GEOTECHNICAL ENGINEERING REPORT

Buttenwieser/Wiley Residence
6838 96th Avenue SE
Mercer Island, Washington

Prepared for: Janet Buttenwieser

Project No. 200631 • September 2, 2021 FINAL

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Aspect
CONSULTING

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Contents

List of Tables

List of Figures

List of Appendices

1 Introduction

This report presents the results of a preliminary geotechnical engineering and critical area evaluation performed by Aspect Consulting, LLC (Aspect) for the proposed new residence (Project) at 6838 96th Avenue SE on Mercer Island, Washington (King County Parcel No. 302405-9010; Site). The Site location is shown on Figure 1.

The purpose of this evaluation is to assess the geologic hazards at the Site, provide recommendations to mitigate impacts, and provide geotechnical engineering conclusions and recommendations to support design and construction of the Project.

1.1 Project Background and Description

The existing Site consists of a single-family residence and detached garage on the southeast side of Mercer Island, adjacent to Lake Washington. The Site is a 0.95-acre lot on a locally steep, east-facing slope accessed via 96th Avenue SE that descends to the Lake Washington shoreline. Our understanding of the proposed improvements is based on communications with the Project architect (Miller Hull Partnership; Miller Hull), Project structural engineer (PCS Structural Solutions; PCS), Project civil engineer (LPD Engineering, LLC; LPD) and our review of permitting-level civil and structural drawings (LPD, 2021; PCS, 2021).

The Project includes demolition of the existing buildings and replacement with a new single-family, three-story residence with a detached garage.

2 Site Conditions

This section presents the surface conditions, geologic setting, and subsurface conditions of the Site, which provides context for the types and distribution of geologic soil units and a basis for our geotechnical engineering recommendations and critical areas evaluation.

2.1 Surface Conditions

Our understanding of the surface conditions is based on a review of publicly available maps and aerial photography, observations made during a Site reconnaissance visit on December 31, 2020, and measurements obtained during our subsurface exploration program completed on February 2 and 3, 2021.

2.1.1 Topography

The Site is an approximately 0.95-acre, rectangular parcel orientated length-wise from east-west. Topography for the Site is presented in Figure 2 from a Site survey by Terrane Land Surveying (2021). The parcel is approximately 100 feet wide in the north-south direction and approximately 400 feet long in the east-west direction. The Site abuts 96th

Avenue SE to the west at approximate Elevation[1] 100 feet and descends steeply at an average slope of approximately 20- to 30-percent to the east and south over approximately 300 horizontal feet to a bench at Elevation 35 feet, which comprises the eastern side of the Site.

The bench slopes over approximately 100 horizontal feet (average approximate slope of 10- to 20-percent) down to the Lake Washington shoreline at approximate Elevation 18 feet. Locally, the Site slopes are highly variable; along the north property line they can exceed 50 percent in the steepest locations. The two existing buildings are accessed from an approximately 200-foot-long concrete driveway that slopes at approximately 5- to 20-percent from 96th Avenue SE to an asphalt parking area near the center of the Site. There is a relatively flat area behind the garage that is used as a garden.

2.1.2 Existing Structures

Existing structures including the house, driveway, garage, and rockeries (Figure 2). The existing two-story residence and detached garage were originally constructed in 1934 and appear to consist of typical wood-frame construction and cast-in-place concrete spread footings. The garage is located west of the asphalt parking area at the bottom of the driveway (at approximate Elevation 55 feet). The residence is approximately 150 feet to the east of the garage near the toe of the slope (at approximate Elevation 24 feet) and approximately 47 feet west of the shoreline. We observed no evidence of structural cracking or settlement around the exterior walls or foundations.

2.1.3 Steep Slopes and Retaining Walls

The Site has several existing retaining walls, including an approximately 5-foot-tall soldier pile wall just east of 96th Avenue SE; an approximately 4-foot-tall rockery wall along the north side of the driveway; an approximately 5- to 8-foot-tall rockery wall at the east side of the asphalt parking area; and several timber walls up to approximately 4 feet tall (along the south side of the driveway, the southern property line [southwest of the existing garage], and northwest of the existing residence). There is also an approximately a 2-foot-tall rockery bulkhead along the Lake Washington shoreline.

The steep slope north of the driveway is vegetated with mixed deciduous and coniferous trees and dense underbrush. We did not observe readily apparent evidence of instability or deformations associated with the rockery wall along the north side of the driveway, but we did observe at least one conifer tree with a slightly curved trunk located on the slope immediately northwest of the existing residence. At approximately the same location, we observed localized yielding of the existing timber retaining wall. We also observed yielding of the timber wall on the south side of the driveway behind the garage during our subsurface exploration program. The concrete driveway is deteriorated with several longitudinal cracks.

These observations are all characteristic of localized surficial slope movement that reflect the age and decay of the railroad tie timbers for the timber wall that are beyond their design life and will need to be replaced.

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[1] All elevations were obtained using survey data completed by Terrane Land Surveying (Terrane; 2021) and reference the North American Vertical Datum of 1988 (NAVD88)

2.2 Subsurface Conditions

Our characterization of the subsurface conditions at the Site are based on a review of applicable geologic literature, data obtained from our subsurface explorations, and our knowledge and understanding of the regional geologic setting.

2.2.1 Geology

The most recent geologic map (Troost & Wisher, 2006) shows the Site as being underlain by nonglacial Pleistocene deposits of pre-Olympia age (Qpon), which predate the most recent glacial period (the Fraser glaciation), as well as Holocene-age lake deposits (Ql) and mass-wastage deposits (Qmw). The nonglacial pre-Olympia deposits are further subdivided into coarse-grained (Qponc) and fine-grained (Qponf) units. The mapped surficial geologic units are described as follows:

- **Fine-grained pre-Olympia nonglacial deposits (Qponf):** Silt and clay; hard, may have sandy interbeds, and peat, laminated to massive. The deposits are mapped along the central area of the Site.
- **Coarse-grained pre-Olympia nonglacial deposits (Qponc):** Sand and gravel; very dense, clean to silty, with silt layers and peat. The deposits are mapped along the west area of the Site.
- **Lake deposits (Ql):** Silt and clay; very soft to medium stiff or very loose to medium dense, with local sand layers, peat, and other organic sediments. The deposits are mapped along the east area of the Site including the shoreline.
- **Mass-wastage deposits (Qmw):** Colluvium, soil, landslide debris, and organic matter with indistinct morphology; loose to dense and soft to stiff. The deposits are mapped along the east area of the Site, including the shoreline.

Although not shown on the geologic map, we expected to encounter fill material placed or disturbed as part of the original Site development (fill observations are discussed further in Section 2.2.2 below). In general, our observations during the subsurface explorations were consistent with the geologic map and our expectations, except that we did not encounter lake deposits or clearly delineated mass-wastage deposits.

2.2.2 Stratigraphy

Aspect completed six drilled soil borings on February 2 and 3, 2021 (designated AB-01 through AB-06). We completed each of the borings to approximately 21 feet below ground surface (bgs) using hollow stem auger drilling techniques, with *in-situ* density/consistency testing and sample collection at select depth intervals. The drilling was subcontracted to Geologic Drill Partners, Inc., who completed the work with a miniature drill rig mounted on a tracked, walk-behind Bobcat. The exploration locations are shown on Figure 2. Aspect also subcontracted geotechnical laboratory testing services for moisture content, fines content, particle-size analyses, and Atterberg limits on select soil samples obtained during our field investigation.

Subsurface conditions at the Site were inferred from the completed field investigation, a review of applicable geologic literature, local geologic experience, and geotechnical laboratory testing. A more detailed description of the field exploration methods and exploration logs are presented in Appendix A. Detailed descriptions of the tests and results are presented in Appendix B.

The primary soil units observed in our explorations, presented in stratigraphic order from top to bottom, were fill, weathered pre-Olympia nonglacial deposits, and intact pre-Olympia nonglacial deposits. Consistent with the geologic map, we encountered fine-grained pre-Olympia nonglacial deposits in the eastern portion of the Site near Lake Washington, that transitioned to coarse-grained deposits at higher elevations in the western portion of Site near 96th Avenue SE. The units are described in more detail below.

Fill

We encountered fill consisting of very soft to medium stiff, moist to wet, gray to brown silt with varying proportions of sand (ML) [2] and very loose to medium dense, moist to wet, gray to brown silty sand (SM) in all explorations from the surface to depths of between 7- to 15-feet below ground surface (bgs). At AB-02, located approximately mid-way down the concrete driveway, we also encountered a layer of medium stiff, moist, brown clay (CL) between 7 and 10 feet bgs. We encountered organics, roots, and woody debris at AB-01, AB-04, and AB-05. Based on the observed relative density and moisture content, the fill was likely placed without moisture or compaction control.

The fill can be expected to exhibit low shear strength characteristics, low to moderate permeability, moderate to high compressibility, and high moisture sensitivity.

Weathered Pre-Olympia Nonglacial Deposits

We encountered weathered pre-Olympia nonglacial deposits at AB-01, AB-02, AB-03, and AB-06 consisting of loose to dense, very moist to wet, brown to gray silty sand with varying proportions of gravel (SM) from the bottom of the fill to depths of between 10- to 15-feet bgs. The weathered pre-Olympia nonglacial deposits are similar to the underlying coarse-grained pre-Olympia nonglacial deposits, but we interpret them to be weathered due to their relatively lower density.

The weathered pre-Olympia nonglacial deposits can be expected to exhibit moderate shear strength characteristics, moderate permeability, moderate compressibility, and moderate moisture sensitivity.

Coarse-Grained Pre-Olympia Nonglacial Deposits

We encountered coarse-grained pre-Olympia nonglacial deposits in AB-01 through AB-04 from below the fill or weathered pre-Olympia nonglacial deposits to depths of between 15 to 21 feet bgs consisting of dense to very dense, slightly moist to wet, gray to brown sand with varying proportions of silt and gravel (SM, SP-SM). The coarse-grained pre-Olympia nonglacial deposits were encountered in AB-03 and AB-04 at an approximately 5-foot-thick layer overlying fine-grained pre-Olympia nonglacial deposits. At AB-01 and AB-02 the coarse-grained pre-Olympia nonglacial deposits were encountered to the bottom of the explorations at approximately 21 feet bgs.

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[2] Soils are classified per the Unified Soil Classification System (USCS) in general accordance with the ASTM International (ASTM) Method D2488 Standard Practice of Description and Identification of Soils.

The coarse-grained pre-Olympia nonglacial deposits can be expected to exhibit high shear strength characteristics, low to moderate permeability, low compressibility, and moderate moisture sensitivity.

Fine-Grained Pre-Olympia Nonglacial Deposits

We encountered fine-grained pre-Olympia nonglacial deposits in AB-03 through AB-06 to depths of between 15 to 21 feet bgs consisting of medium stiff to hard, slightly moist, gray clay (CH). We interpret this clay as being highly overconsolidated and relatively intact and undisturbed (i.e., we did not observe significant evidence of fracturing, slickensides, or shearing).

The fine-grained pre-Olympia nonglacial deposits can be expected to exhibit high shear strength characteristics, low permeability, low compressibility, and moderate to high moisture sensitivity.

2.2.3 Groundwater

Groundwater was encountered in boring AB-01, where it was measured at a depth of 5.9 feet bgs at the time of drilling. The apparent moisture content of the samples in AB-06 suggest that there may have been some perched groundwater in the weathered pre-Olympia deposits at approximately 8 feet bgs above the relatively impermeable, fine-grained pre-Olympia nonglacial deposits. Red mottling and iron oxide staining was observed in several of the samples over a wide range in depths, which can indicate seasonal fluctuations in groundwater levels. We expect the groundwater on the slope is in hydraulic continuity with Lake Washington. Groundwater levels are expected to fluctuate by seasonal conditions, Site usage, variations in rainfall, irrigation, and other factors.

3 Geologic Hazard Evaluation

Erosion, sliding, and earthquake hazard areas are geologically hazardous areas as defined in Sections 19.16 of the Mercer Island City code (MICC; 2021). Development on the Site is therefore governed by the requirements of MICC 19.07. This report is intended to serve as the required critical area study to describe existing conditions, potential impacts, and risk mitigation measures consistent with MICC 19.07.110 and 19.07.160.

As part of our evaluation, we reviewed publicly available critical area maps relative to geologic hazards, as shown on Figure 2. The City of Mercer Island maps the entire parcel as a potential slide hazard area and as an erosion hazard area. The majority of the Site is also mapped as a seismic hazard area, and localized areas in the north portion of the Site are mapped as steep slope hazard areas. A historic landslide scarp is mapped on parcels immediately south of the Site (Troost and Wisher, 2006).

3.1 Landslide / Steep Slope Hazards

As part of our landslide / steep slope hazard evaluation, we reviewed the Site topography, landslide map inventories, and historic aerial photographs from 1936 and 2019 (King County, 2021). Steep slopes are defined by the City as any slope exceeding 40 percent over a 30-foot horizontal run. Based on a recent Site survey completed by Terrane Land Surveying (Terrane, 2021), steep slopes are present on the slope north of the driveway and west of the garden behind the garage. We previously described some localized slope movement associated with decaying timber walls along steep slopes. In general, we observed no indications of global slope movement from our reconnaissance or review of aerial photographs from 1936 to 2019.

Three types of landslides hazards are common for slopes in the Puget Sound region:

- Rotational (deep-seated) landslides
- Shallow landslides
- Topping failures.

Landslides may be triggered by natural causes such as precipitation, freeze-thaw cycles, or earthquakes, or by man-made events such as broken water pipes or stormwater flow. Each of these landslide hazards is discussed in greater detail below with respect to the Site.

3.1.1 Rotational Landslides

Rotational landslides consist of deep-seated failures that are characterized by slip along a curved shear plane. Rotational landslides may transport larger masses of semi-intact soil downslope, resulting in steep head scarps along the upper portion of the failure plane, and benches and hummocks of displaced soil lower on the slope. Rotational landslides can be caused by ongoing processes, such as erosion of the toe of the slope, seeps and springs on the steep slope, and other ongoing processes. Deep-seated (below rooting depth for trees) rotational landslides can also be triggered by large earthquakes.

Deep-seated landslides can cause significant damage because of the volume of soil that they can displace. However, these landslides typically don’t occur without warning signs many days in advance, such as formation of open tension cracks at the ground surface, slow downslope creep of soils, bending and tipping trees, displacement of infrastructure, etc.

Based on our reconnaissance and the dense, high-shear strength of the glacially consolidated deposits that comprise the core of the Site slopes, it is our opinion that the risk of large-scale, deep-seated rotational landslide activity is low.

3.1.2 Shallow Landslides

Shallow landslides consist of sliding of the surficial, colluvial, or weathered soil layers and overlying vegetation that typically mantle steep slopes in the Puget Sound region. Shallow landslides are commonly triggered by a significant increase in the moisture content within the upper soil layers of a slope combined with a slow increase in the thickness of weathered and loose surficial soils over geologic time. Increased moisture typically results from periods of extended, heavy precipitation, groundwater seepage, or concentrated surface water discharge onto a slope.

While shallow landslides displace a smaller volume of soil than deep-seated rotational landslides, they can be fast moving and can occur with little or no warning. Shallow slides are typically less than five to ten feet thick and several tens of feet in width. They typically do not extensively impact the underlying denser soils or affect overall stability of a slope beyond the area that has slid.

Based on our review of the Site topography and vegetation, the presence of mapped mass wastage deposits, and our observations and experience with slopes in the Puget Sound region, we assess the potential for shallow landslides at the Site to be moderate. The potential for shallow landslides increases following extended periods of heavy precipitation or during a seismic event.

3.1.3 Toppling Failures

Toppling failures involve a mass of soil peeling off along naturally occurring tension cracks, which form in soils at the crest of steep slopes and bluffs. These tension cracks may provide conduits for surface water migration and flow, and they also promote growth of tree roots that can extend many feet downward into the cracks. As the roots grow and the face of the slope progresses through freeze-thaw cycles, or when the face of the slope at the toe of the tension crack becomes oversteepened and undermined by erosion, these cracks often become failure planes, and a slab of soil will spall or topple off the slope face. Failures of this kind are typically not more than several feet thick and occur only on very steep to near-vertical sections of slopes.

In our opinion, the potential for toppling failures at the Site is low.

3.1.4 Landslide Hazard Summary

The existing conditions include pipes, catch basins, and conveyance to an outfall at Lake Washington to manage drainage and reduce the risk for landslides. Drainage at the Site should be maintained or enhanced as part of the redevelopment to mitigate the potential for future landslide and steep slope hazards. Areas south of the driveway and west of the garage need drainage improvements to reduce the risk for instability in the vicinity of the timber walls observed during explorations and our reconnaissance.

The proposed redevelopment will occur in previously graded or developed areas of the house, garage, driveway, sod-surfaced areas between the house and driveway, and parking areas that were originally developed in 1934. The areas proposed for redevelopment are generally stable and have performed as intended. Provided Site development recommendations in this report are followed, the proposed development will, in our opinion, not pose a threat to the public health, safety, and welfare due to geologic hazards.

3.2 Erosion Hazards

We did not observe evidence of substantial erosion, scour, or rilling at the Site. Care should be taken during construction to mitigate risks of erosion. Appropriate temporary erosion and sedimentation control (TESC) best management practices (BMPs) should be implemented in accordance with City requirements.

The existing conditions include pipes, catch basins and conveyance to an outfall to Lake Washington at the Site to manage drainage and reduce the risk for erosion. Drainage at the Site should be maintained or enhanced going forward to mitigate erosion hazards. The proposed development will occur in previously graded or developed areas of the house, garage, driveway, and parking areas that are currently managed to reduce erosion and have performed as intended. Provided Site development recommendations in this report are followed, the proposed development will, in our opinion, not pose a threat to the public health, safety and welfare due to erosion hazards.

3.3 Seismic Hazards

The Site is located within the Puget Lowland physiographic province, an area of active seismicity that is subject to earthquakes on shallow crustal faults and deeper subduction zone earthquakes. The Site lies within the Seattle Fault Zone (SFZ; Troost and Wiser, 2006), which consists of shallow crustal tectonic structures that are considered active (evidence for movement within the Holocene [since about 15,000 years ago]) and are believed to be capable of producing earthquakes of magnitude 7.3 or greater. The recurrence interval of earthquakes on this fault zone is believed to be on the order of 1,000 years or more. The most recent large earthquake on the SFZ occurred about 1,100 years ago (Pratt et al., 2015). Thrust fault traces are mapped approximately 4,700 feet north and approximately 2,300 feet south of the Site. Several other shallow crustal faults in the region are also capable of producing earthquakes and strong ground shaking.

The Site also lies within the zone of strong ground shaking from earthquakes associated with the Cascadia Subduction Zone (CSZ). Subduction zone earthquakes occur due to rupture between the subducting oceanic plate and the overlying continental plate. The CSZ can produce earthquakes up to magnitude 9.3 and the recurrence interval is thought to be on the order of about 500 years. A recent study estimates the most recent subduction zone earthquake occurred around 1700 (Atwater et al., 2015).

Deep intraslab earthquakes, which occur from tensional rupture of the sinking oceanic plate, are also associated with the CSZ. An example of this type of seismicity is the 2001 Nisqually earthquake. Deep intraslab earthquakes typically are magnitude 7.5 or less and occur approximately every 10 to 30 years.

Mitigation design to address seismic hazards will be incorporated into the development plans based on the following sections to prevent increased risk of harm to life and/or property.

3.3.1 Seismic Design Parameters

Seismic design of the improvements will be in accordance with the 2018 International Building Code (IBC), which references the American Society of Civil Engineers (ASCE) Standard ASCE/SEI 7-16, Minimum Design Loads for Buildings and Other Structures (ASCE, 2018) for seismic design. In accordance with these codes, the seismic design will consider a “Maximum Considered Earthquake” (MCE) ground motion with a 2 percent probability of exceedance in 50 years, or a return period of 2,475 years.

The effects of Site-specific subsurface conditions on the MCE ground motion at the ground surface are determined based on the “Site Class.” The Site Class can be correlated to the average standard penetration resistance (N-value), average shear wave velocity, or average undrained strength (for fine-grained soils) in the upper 100 feet of the soil profile. Based on the average N-value from our explorations, we conclude the Site soil profile can be classified as Site Class D (Stiff Soil).

The design spectral response acceleration parameters adjusted for Site Class D in accordance with the 2018 IBC and ASCE/SEI 7-16 are presented in Table 5. These parameters are only valid if the exceptions outlined in Section 11.4.8 of ASCE/SEI 7-16 are met. If the exceptions are not met, then a Site Response Analysis in accordance with Section 21.1 of ASCE/SEI 7-16 is necessary. If the need for a Site Response Analysis becomes apparent as the Project design develops, Aspect can complete this upon request.

Table 1. Seismic Design Parameters

Design Parameter	Recommended Value
Site Class	D – Stiff Soil(1)
Peak Ground Acceleration (PGA)	0.620g(2)
PGA Coefficient (F_{PGA})	1.1
Site Modified PGA (PGA_M)	0.682g
Short Period Spectral Acceleration (S_s)	1.449g
1-Second Period Spectral Acceleration (S_1)	0.501g
Site Coefficient (F_a)	1.0
Site Coefficient (F_v)	1.8
Design Short Period Spectral Acceleration (S_{DS})	0.966g
Design 1-Second Period Spectral Acceleration (S_{D1})	0.601g

Notes:

1. Verify that the exceptions outlined in Section 11.4.8 of ASCE/SEI 7-16 are met. Refer to text above
2. g = gravitational force
3. Based on the latitude and longitude of the Site: 47.541180°N, -122.210110°W.
4. The risk category used was II, residential use.

3.3.2 Liquefaction

Liquefaction occurs when loose, saturated, and relatively cohesionless soil deposits temporarily lose strength from seismic shaking. The primary factors controlling the onset of liquefaction in susceptible soils include intensity and duration of strong ground motion, *in situ* stress conditions, and the depth to groundwater.

We evaluated the susceptibility of the Site soils to liquefaction based on geologic, compositional, and state criteria. The Washington Department of Natural Resources (DNR) maps the Site as generally having low to moderate liquefaction susceptibility (DNR, 2004). The loose, surficial fill deposits overlying the Site are potentially susceptible to liquefaction. This is due to their low density and because the fine-grained particles are relatively nonplastic. Liquefaction would only be expected to initiate in the fill deposits under saturated conditions, which were not observed during our subsurface explorations. In addition, the laboratory analysis results on select samples suggest that the fines content in the fill materials is on the order of approximately 15 percent or more, which may inhibit the initiation of liquefaction.

In our opinion there is some risk of liquefaction initiating in the fill deposits during the life of the Project, if saturated conditions coexist with strong ground shaking. To mitigate this risk, we have recommended deep foundation alternatives that will bypass the fill deposits and bear the structures on pre-Olympia nonglacial deposits. It is our opinion that the pre-Olympia nonglacial deposits are not susceptible to liquefaction due to their high density. Based on the reasoning presented above, we do not expect liquefaction to be a significant hazard for the Project.

3.3.3 Surface Fault Rupture

The SFZ passes directly through Mercer Island. The U.S. Geological Survey maps east-west trending traces approximately 1 mile north and approximately 0.5 miles south of the Site (USGS, 2016). Due to the suspected long recurrence intervals and the proximity of the Site to the mapped fault traces, the potential for surficial ground rupture at the Site itself is considered low during the expected life of the Project.

4 Geotechnical Conclusions and Recommendations

Based on our evaluation, the Project is feasible from a geotechnical perspective. A summary of key Project geotechnical conclusions and recommendations are listed below and described in more detail in the following sections.

- Relatively compressible and low-strength fill deposits overlie the Site to depths of between 7- to 15-feet bgs. In order to mitigate risks to the proposed structures from differential settlement, we recommend that the structures be founded on deep foundations that bypass the fill and bear on the dense, high-strength pre-Olympia nonglacial deposits beneath the fill. Estimates of foundation capacities and design and construction recommendations for these foundation systems are included in subsequent sections.
- The Project will include new retaining walls, including cantilevered soldier pile and lagging wall systems and cast-in-place cantilevered concrete walls. Estimates of lateral earth pressures, global stability evaluations, and other wall design and construction recommendations are provided in subsequent sections.
- The existing concrete driveway has failed and will require replacement. We understand this will occur in a subsequent phase of construction. We have provided recommendations for flexible and rigid pavement sections that will mitigate risk of premature failure over the design life of the pavement due to the soft subgrade.
- The surficial fill deposits are moisture sensitive and generally not suitable for reuse as structural fill.

4.1 Soil Engineering Properties

The engineering properties of the subsurface soils were generalized for engineering analysis purposes. These parameters are shown for each observed geologic unit in Table 2. These values serve as the basis for our geotechnical recommendations and conclusions and can be used by the Project structural engineer directly to evaluate design scenarios that we have not explicitly considered in this report.

Table 2. Soil Engineering Properties

Soil Unit	USCS Classification	SPT N-Value(1)	Total Unit Weight (pcf)[2]	Effective Friction Angle (degrees)	Effective Cohesion Intercept (psf)[3]
Fill	SM, ML, CL	R: 1-14 A: 7	110	30	-
Weathered Pre-Olympia nonglacial	SM	R: 8-37 A: 25	125	35	-
Coarse-Grained Pre-Olympia nonglacial	SM, SP-SM	R: 40-90 A: 66	135	40	-
Fine-Grained Pre-Olympia nonglacial	CH	R: 6-41 A: 24	130	30	500

Notes:

1. Uncorrected. R = range, A = average
2. Pounds per cubic foot, pcf
3. Pounds per square foot, psf

4.2 Building Foundations

In our opinion, the compressible surficial fill deposits are unsuitable for conventional shallow foundations due to the risks from differential settlement. To mitigate these risks, we recommend that the new structures be founded on deep foundations that bypass the fill deposits and gain capacity from the underlying pre-Olympia nonglacial deposits. The use of deep foundations at the Site has the secondary benefit of mitigating the more moderate risks from liquefaction or shallow slope failures in the fill deposits.

During the preliminary design phase, we evaluated both helical and pin pile foundation alternatives. We understand that the design team has elected to use pin piles, so we have included appropriate recommendations for pin pile design and construction below.

4.2.1 Pin Piles

For residential foundation support, pin piles typically consist of 2- to 6-inch-diameter steel pipe piles driven to a predetermined acceptance criterion using a pneumatic or hydraulic hammer. Acceptance criteria varies by the diameter of the pin pile but are typically defined as less than 1 inch of penetration into the ground during a specified time period of continuous driving with the specified hammer. Specific acceptance criteria and allowable load capacity information is shown below in Table 3.

Table 3. Typical Pin Pile Capacities and Installation Acceptance Criteria

Pin Pile Diameter (in)	Hammer Weight$^{(1)}$ (lbs)	Allowable Capacity$^{(2)}$ (kips)	Acceptance Criteria$^{(3)}$ (sec)
2	90	4	60
3	550	12	12
4	850	20	16
6	2,000	30	10

Notes:

1. Minimum hammer weight recommended
2. Includes a factor of safety of 2
3. Time to drive pile less than 1 inch during continuous driving

Pin pile spacing, lateral requirements, and structural connections to other foundation elements should be designed by the Project structural engineer. We recommend schedule 80 or XS pipes for 2-inch-diameter piles and galvanized, schedule 40 pipes for 3- to 6-inch-diameter piles.

Pin piles should be utilized for axial, compressive support only. If lateral resistance is required, the pin piles may be installed on a slight batter (10 to 20 degrees from vertical) and the horizontal component of their axial capacity may be assigned as lateral resistance. This horizontal capacity will be available only in the direction of batter.

The capacities of piles greater than 2 inches in diameter should be verified through load testing in general accordance with the *Quick Load Test Method* described in ASTM D1143 (ASTM, 2018). We recommend a minimum of two piles be load tested in different areas of the proposed residence footprint prior to installing the production piles for the Project. The test piles may be incorporated as production piles at the discretion of the geotechnical engineer, provided they successfully pass the load test and are not damaged during installation or load test.

The pin piles should be required to extend to a minimum of 3 feet into the pre-Olympia nonglacial deposits (to be estimated based on observations during pile driving). Based on our explorations, we estimate that the total pile lengths to achieve the acceptance criteria shown in Table 2 will be on the order of approximately 15 feet in the vicinity of the main residence and approximately 25 feet in the vicinity of the garage. Due to buckling considerations, 2-inch-diameter pin piles shall not exceed 30 feet in length.

4.2.2 Foundation Lateral Resistance

We recommend that lateral resistance from pin piles be neglected unless they are battered. Passive and frictional resistance against pile caps/grade beams and below-grade walls can be considered for lateral resistance. Assuming the foundation elements are constructed within the existing fill deposits, we recommend using a passive equivalent fluid density of 350 pounds per cubic foot (pcf). A base friction coefficient of 0.30 may be used to evaluate sliding resistance developed between concrete and the compacted subgrade soil. These values include a factor of safety of 1.5. Passive resistance within the top foot should be neglected unless the ground surface is protected by a concrete slab or pavement.

4.2.3 Floor Slabs

We recommend that the new structures be founded on deep foundations that bypass the surficial fill deposits. In our opinion, floor slabs that are not structurally integrated to the deep foundation system are feasible for floor loads up to 150 psf, provided the subgrade is prepared in accordance with our recommendations. Specifically, we recommend that the subgrade below floor slabs be overexcavated to a minimum depth of 18 inches and replaced with structural fill compacted to at least 95 percent of the maximum dry density determined by the modified Proctor. Additional overexcavation may be necessary if deleterious, organic, wet, or oversized material is encountered. Prior to placing the structural fill, the subgrade surface should be compacted to a firm and unyielding condition.

For floor slabs that are not structurally integrated with the deep foundation system, it should be understood that some risk of concrete distress exists due to the potential for future settlements. Future maintenance associated with this risk may be required.

For slabs-on-grade designed as a beam on elastic subgrade, we recommend using an initial vertical modulus (K_{v1}) of 120 pounds per cubic inch (pci). The K_{v1} value is appropriate for a 1-foot by 1-foot slab and needs to be adjusted based on the actual width (B) of the slab to a design vertical modulus (Ks) using the following equation below:

$$K_s = K_{v1}(B+1)^2/(4B^2),$$

where B = slab width (in feet).

Alternatively, pile-supported, structural floorslabs can be designed and constructed to mitigate risk of concrete distress from potential settlement.

For interior slabs-on-grade, we recommend the uppermost 6 inches of the subgrade consist of compacted capillary break material (in lieu of 6 inches of crushed surfacing base course [CSBC]) to provide uniform support and moisture control. The capillary break material should consist of free-draining, clean, fine gravel and coarse sand with a maximum particle size of about 1-inch and less than 3 percent material passing the U.S. No. 200 sieve by weight (fines). Angular material manufactured by crushing is preferred over rounded material such as bank run sand and gravel, to provide a subgrade surface that is not easily disturbed by workers laying steel rebar and concrete formwork. The capillary break material should be compacted to relatively firm and unyielding condition and evaluated by Aspect prior to placement of steel rebar and formwork.

For building areas where vapor intrusion mitigation would be detrimental to the interior finished space (such as air-conditioned office areas that may be covered with flooring), consideration should be given to placement of a vapor barrier over the capillary break. Detailed design and performance issues with respect to vapor intrusion and moisture control as it relates to the interior environment of the structure are beyond the expertise of

Aspect. A building envelope specialist or contractor should be consulted to address these issues, as needed.

4.2.4 Settlement

Total and differential static settlement of the structures are anticipated to be less than 0.5 inch, if founded on pin piles or helical piles installed in accordance with our recommendations provided above. Any static settlement is anticipated to occur rapidly as the structural loads are applied during construction.

4.3 Retaining Walls

Based on discussions with the design team and our review of preliminary design documents, we identified three primary retaining walls at the Site:

- **Wall 1:** cast-in-place concrete wall located along the southern property line south of the garage
- **Wall 2:** cast-in-place concrete wall located along the south side of the driveway west of the garage
- **Wall 3:** cantilevered soldier pile wall located at the bottom of the Environmentally Critical Area (ECA) steep slope north of the main residence

These walls, as well as preliminary grading information provided by the design team, are shown in Appendix C-1. The following sections contain design and construction recommendations for proposed retaining walls. All proposed retaining walls should be designed by the Project structural engineer.

4.3.1 Lateral Earth Pressures

Lateral earth pressures acting on earth retaining systems with assumed geometries for active, at-rest, and seismic conditions are shown below in Table 4. The equivalent seismic earth pressure is based on pseudo-static analysis applying a horizontal acceleration of one half of the site-modified PGA from Table 1. These values assume that new walls will primarily retain existing fill deposits at an approximately vertical interface. These values also assume that existing fill deposits will provide passive support in front of the structures. To invoke active earth pressure conditions, a wall must be capable of yielding laterally at least 0.001 to 0.002H, where H is the exposed height of the wall; otherwise, at-rest conditions should be assumed.

Table 4. Lateral Earth Pressure Parameters

Earth Pressure Condition	Foreslope Condition	Backslope Condition	Earth Pressure Coefficient	Equivalent Fluid Density[2] (pcf)[1]	Uniform Lateral Surcharge Pressure[3] (psf)[1]
Active	-	Level	0.33	40	0.33S
Active[4]	-	2H:1V	0.52	63	0.52S
Passive[5]	Level	-	3.20	350	-
Passive[4,5]	2H:1V	-	0.90	110	-
At-Rest	-	Level	0.50	60	0.50S
Seismic	-	Level	-	-	18.0H

Notes:

1. psf = pounds per square foot; pcf = pounds per cubic foot.
2. The equivalent fluid densities provided above are distributed triangularly along the exposed height of the wall. The uniform lateral surcharge pressures are distributed uniformly (rectangularly) along the exposed height of the wall.
3. S is the vertical surcharge pressure at the ground surface immediately above/behind the wall. H is the height of the wall. The resultant uniform rectangular lateral pressure should be applied to the full height of the wall.
4. These values assume a maximum backslope/foreslope of 2H:1V. Linear interpolation can be used for shallower backslope/foreslope conditions.
5. The passive value includes a factor of safety of 1.5. Passive resistance within a depth of 2 feet of the ground surface in front of the walls should be ignored.

4.3.2 Wall Global Stability

The purpose of our global stability analyses was to calculate factors of safety against global failure and determine minimum recommended embedment for the soldier piles (for the soldier pile wall) and/or wall footings (for the precast concrete walls) to ensure global stability. We performed global stability analyses for the proposed walls using topographic survey data and proposed grading information provided by the design team, as well as the results of our subsurface exploration program. We selected critical cross section locations for our analyses based on the expected locations of the maximum heights of the walls, as shown in Appendix C-1.

We conducted two-dimensional limit equilibrium slope stability analyses (SSA) using the Slide computer software program (Rocscience, 2018). We assessed stability under both static and seismic conditions. The Slide program performs slope stability computations based on the modeled slope conditions and calculates a factor of safety against slope failure, which is defined as the ratio of resisting forces to driving forces. A factor of safety of 1.0 indicates a “just-stable” condition, and a factor of safety less than 1.0 would indicate unstable conditions. Minimum factors of safety of 1.5 and 1.1 for static and seismic loading conditions, respectively, are generally considered acceptable.

We designated the soil/material units and assigned the engineering parameters shown in Table 2 and modeled a groundwater surface perched atop the fine-grained pre-Olympia nonglacial deposits and saturating the coarse-grained pre-Olympia deposits. We made the following specific assumptions regarding wall geometry at each wall location (refer to Appendix C-1 for wall locations):

Wall 1 – located along the southern property line south of the garage:

- Wall Type: Cast-in-place concrete
- Maximum Exposed Height: 5.5 feet
- Minimum Footing Embedment: 3 feet

Wall 2 – located along the south side of the driveway west of the garage:

- Wall Type: Cast-in-place concrete
- Maximum Exposed Height: 4 feet
- Minimum Footing Embedment: 3 feet

Wall 3 – located at the bottom of the ECA steep slope north of the main residence:

- Wall Type: Cantilevered soldier piles with lagging
- Maximum Exposed Height: 4 feet
- Soldier Pile Spacing: 8 feet
- Ultimate Pile Shear Strength: 160 kips
- Minimum Pile Embedment: 8 feet[3]

The model inputs, geometry, and results are presented graphically in Appendix C-2 through C-11. The calculated factors of safety for global stability are summarized in Table 5 below, which meet or exceed the recommended minimums in each case. Our analyses indicate that minor surficial sloughing should be anticipated during the design seismic event in isolated areas on some of the existing steep slopes. These locations are not anticipated to be graded or otherwise disturbed as part of the Project. In our opinion, these surficial areas should be considered maintenance issues and are not indicative of global instability for the retaining walls.

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[3] We recommend that the soldier piles penetrate the minimum embedment recommended above, or a minimum of 1 foot into the fine-grained Pre-Olympia nonglacial deposits, whichever is deeper. Thus. the minimum embedment depth should be established in the field based on observations during construction.

Table 5. Summary of Factor of Safety Values for SSA Results

Wall ID	Analysis Cross Section	Static Factor of Safety for Global Stability(1)	Seismic Factor of Safety for Global Stability (2)
1	A-A’	1.1	2.0
2	B-B’	1.1	2.1
3	C-C’	1.1	2.2
3	D-D’	1.1	2.4
3	E-E’	1.1	2.2

Notes:
1. Limit equilibrium minimum factor of safety found using Spencer’s method in SLIDE
2. Pseudostatic seismic analysis with a horizontal seismic coefficient of 0.341g

4.3.3 Wall Drainage

Drainage behind walls should consist of a 24-inch-thick zone of free-draining sand and gravel meeting the requirements for WSDOT Standard Specification 9-03.12(2) for Gravel Backfill for Walls. A woven geotextile separator meeting the requirements of Section 9-33.2(1), Table 3 of the WSDOT Standard Specifications should be included at the interface between the native soils and the drain rock behind the walls. Water that is carried down by this sand and gravel zone should be conveyed to a drainage system consisting of a minimum 4-inch-diameter, perforated, Schedule 40 PVC pipe surrounded by at least 6 inches of washed gravel meeting the requirements for WSDOT Standard Specification 9-03.12(4) for Gravel Backfill for Drains. The drain should be routed to discharge at an appropriate location with positive drainage away from the wall.

4.3.4 Wall Bearing Resistance

Precast concrete walls can bear on the fill deposits if the subgrade is suitably prepared and improved with a 12-inch-thick crushed rock fill pad (fill pad) composed of CSBC per WSDOT Standard Specification 9-03.9(3) (WSDOT, 2021). The compacted CSBC pad should be placed over firm and unyielding soil. We estimate that foundation widths in this application will be on the order of 1 to 5 feet wide. We recommend a maximum allowable bearing pressure of 1,500 psf be used for design to limit settlements. An increase in the allowable bearing pressure of one-third may be used for transient loading (e.g., wind, seismic). Lateral resistance along the base of wall foundations can be calculated with an allowable coefficients of friction of 0.30, which assumes a factor of safety of 1.5.

4.4 Driveway Pavements

The fill deposits are expected to provide relatively poor structural support for new pavement. Even though traffic loading is expected to be low, we recommend a robust pavement section. For flexible, hot mix asphalt (HMA) pavement surfaces, we recommend a section consisting of 3 inches of HMA overlying 8 inches of crushed surfacing. For rigid, unreinforced concrete surfaces, we recommend minimum 6 inches of concrete overlying 6 inches of crushed surfacing. Compaction requirements are discussed in detail in Section 5.1.3

4.5 Steep Slope Management

Many of the factors that can cause landslides, such as site geology, topography, and groundwater conditions cannot be controlled. Some factors such as vegetation and stormwater runoff, however, can be controlled, and homeowners are advised to maintain the Site in a manner that maximizes slope stability.

The most likely impact to the Site from a slope stability perspective would be shallow landslides caused by saturation of the surficial fill soils on the steep slope, or from inertial forces during a seismic event. Factors that affect slope stability within the near-surface soil layer include the following (Gray and Leiser, 1982):

- **Root Reinforcement –** Roots mechanically reinforce a soil by transfer of shear stresses in the soil to tensile resistance in the roots.
- **Soil Moisture Modification –** Evapotranspiration and interception in the foliage limit buildup of soil moisture.
- **Buttressing and Arching –** Anchored and embedded stems can act as buttress piles or arch abutments in a slope, counteracting shear stresses.
- **Surcharge –** Weight of vegetation on a slope exerts both a downslope (destabilizing) stress and a stress component perpendicular to the slope, which tends to increase resistance to sliding.
- **Root Wedging –** Alleged tendency of roots to invade cracks, fissures, and channels in a soil or rock mass and thereby cause local instability by a wedging or prying action.
- **Windthrowing –** Destabilizing influences from an overturning moment exerted on a slope as a result of strong winds blowing downslope through trees.

Root reinforcement, soil moisture modification (reduction), and buttressing and arching will increase surficial slope stability at the Site. Surcharge, root wedging, and windthrowing will have a destabilizing effect on surficial slope stability.

Other sources of surficial slope instability include improperly managed storm and surface water runoff flowing near or over the top of the slope. Uncontrolled runoff or surface water should never be allowed to flow across the slope.

Care should be taken not to over-irrigate near the slope. If an irrigation system is installed near the steep slope, we recommend you install a shutoff valve well away from the slope and shut the valve during the wet season. This will reduce the risk of flooding of the hillside due to pipe damage. We recommend limiting irrigation to the dry season (between April and October).

To minimize soil erosion and reduce the risk of shallow landslides, we recommend establishing/ maintaining dense native vegetative cover that is low and has deeply-penetrating roots. We recommend consulting with a professional landscaper to determine appropriate vegetation types and to develop a planting plan for any steep slopes that are disturbed during construction. Grading activities on the Site slopes that do not result in increased slope stability (i.e., placement of fill to flatten the slope) should be minimized to the maximum extent practical. If required, disturbance should be minor (limited to the outer 12 inches of the slope), accomplished with hand tools, and should facilitate replanting and promote vegetative growth. Grading activities should not result in a steeper inclination of the slope or the placement of new fill at the top of the slope. Landscaping debris should not be placed on the steep slope as this inhibits the growth of beneficial vegetation and adds mass to the surficial soil layers.

If soils on or near the steep slope become exposed through erosion and/or surficial landslide activity, we recommend immediately covering and aggressively revegetating the exposed areas. This may require the temporary placement of plastic sheeting replaced during the spring by a woven jute-mat (erosion control blanket) to provide temporary ground cover while vegetation takes root.

For specific vegetation recommendations, the Washington State Department of Ecology (Ecology) has several good publications on the subject including:

- Vegetation Management: A guide for Puget Sound Bluff Property Owners (Ecology, 1993a).
- Slope Stabilization and Erosion Control Using Vegetation: A Manual of Practice for Coastal Property Owners (Ecology, 1993b).

This information is also available from Ecology’s website, along with a steep-slope planting guide.

5 Construction Recommendations

5.1 Soldier Pile Wall Construction

The soldier piles must be properly constructed to perform as designed. The soldier pile wall should be constructed in accordance with the applicable portions of Section 6-16 of the WSDOT Standard Specifications (WSDOT, 2021). We recommend the following:

- Groundwater and caving soil could be encountered during drilling of soldier pile shafts, and the contractor should be prepared to use a temporary casing or drilling slurry to prevent caving and soil loss. If there is standing water or drilling slurry in the shaft, concrete should be placed with a tremie pipe placed at the bottom of the hole.
- Boulders and/or cobbles could be present in the subsurface soils. The Contractor should be prepared to remove, break-up, cut through, or otherwise manage obstructions, if encountered.
- Soldier piles with center-to-center spacing of less than 3 pile-hole diameters should not be drilled in sequence. Rather, every other pile should be drilled, and

the concrete should be placed and allowed to cure at least 24 hours before adjacent piles are drilled.

- The bottom of the soldier pile shafts should be cleared of loose or slough soils that may have accumulated during drilled prior to installing the soldier pile.

Aspect should provide special inspection services during soldier pile installations, to include monitoring pile shaft drilling, acceptance of the pile shafts, and inspection of the pile and concrete installation. Acceptance of the soldier pile installation should be the responsibility of the geotechnical engineer.

5.2 General Earthwork Recommendations

Based on the materials encountered in the explorations and our understanding of the Project, we anticipate Site earthwork can be completed with standard construction equipment. Toothed buckets may be required for excavations within the coarse-grained pre-Olympia nonglacial deposits. The construction of temporary gravel access roads and working platforms may also be required to navigate the Site. Appropriate erosion and sedimentation control measures should be in accordance with local BMPs and should be implemented prior to beginning earthwork activities. Also, land clearing, grading, filling, and foundation work within the identified geologic hazard areas are not permitted between October 1 and April 1.

5.2.1 Temporary Excavations

Temporary excavation and slopes should not exceed the limits specified in the local, state, and federal regulations. Site Safety, including the stability of temporary excavations and slopes shall be the responsibility of the contractor. The soils within the anticipated excavation depths would classify as Type C soils in accordance with the Washington Administrative Code (WAC) 296-155 Part N (WAC, 2016). For planning purposes, we recommend that temporary slopes in Type C not be steeper than 1.5H:1V (horizontal to vertical). The presence of seepage may require that slopes be flattened further to remain stable.

We also recommend the following:

- Surface water should be diverted away from slopes.
- Protect slopes using plastic sheet, flash coating, or tarps to control erosion and stability, as necessary.
- Limit the duration that excavations or slopes are open to the shortest time possible.
- Traffic, equipment, and material stockpiles should not be allowed near the top of excavations or slopes.

The conditions of the excavations and slopes should be periodically observed by a competent person who is a representative of the contractor, to evaluate safety and stability.

5.2.2 Subgrade Preparation

Prior to placing structural fill or constructing foundations, subgrades should be prepared to a relatively firm and level condition that is generally free of standing water and protruding cobbles and compacted until firm and unyielding with appropriate equipment. An Aspect geotechnical engineer or geologist should evaluate foundation subgrades to verify conditions.

5.2.3 Structural Fill

Soils placed beneath or around foundations, fill embankments, walls, utilities, or below pavements should be considered structural fill. For these areas, we provide the following recommendations:

- Site-derived soils are generally unsuitable for reuse as structural fill due to their high fines (material passing the U.S. No. 200 sieve) content and moisture sensitivity.
- Structural fill below foundations and pavements should consist of crushed rock meeting the requirements for WSDOT Standard Specification 9-03.9(3) for CSBC.
- Structural fill directly behind walls should consist of sand and gravel meeting the requirements for WSDOT Standard Specification 9-03.12(2) for Gravel Backfill for Walls.
- Structural fill for utility bedding and backfill should meet the requirements for WSDOT Standard Specification 9-03.12(3) for Gravel Backfill for Pipe Zone Bedding or the material specified in the Standard Specification section applicable to the type of pipe being installed.
- Structural fill should only be placed on a relatively firm and unyielding subgrade.
- Structural fill should be compacted to a relatively firm and unyielding condition to a minimum density of 95 percent of the material maximum dry density as determined by ASTM D1557. Structural fill placed behind walls should be compacted to between 90 to 92 percent of the maximum dry density to avoid overstressing the walls.
- Structural fill should be placed in lifts with a loose thickness no greater than 12 inches when using relatively large compaction equipment, such as a vibrating plate attached to an excavator (hoe pack) or drum roller. If small, hand-operated compaction equipment is used to compact structural fill, lifts should not exceed 6 inches in loose thickness.
- Moisture content of the structural fill should be controlled to within 2 to 3 percent of the optimum moisture. Optimum moisture is the moisture content corresponding to the maximum modified proctor dry density.
- Fill placed in softscape, general grading, landscape, or common areas that are not beneath or around structures, utilities, slabs-on-grade, or below paved areas that can accommodate some settlement should be compacted to a relatively firm and unyielding condition.

5.2.4 Temporary Erosion and Sedimentation Control

Temporary erosion control measures should be implemented to prevent the migration of soil, dust, and turbid water off-Site or into stormwater systems. Such measures should include silt fences and straw wattles at the Site boundaries, silt socks in nearby catch basins, wetting exposed soil during dry periods, and quarry spalls and wheel wash stations at truck and equipment exits.

5.2.5 Wet Weather Construction

Performing Site earthwork during dry summer months is preferred, but the following considerations should be incorporated into the Project requirements in the case that work is completed during wet weather.

- Earthwork should be performed in small areas to minimize exposure to wet weather.
- Excavation or the removal of unsuitable soils should be followed promptly by the placement and compaction of clean structural fill.
- The size and type of construction equipment used may have to be limited to prevent soil disturbance.
- The ground surface within the construction area should be graded to promote runoff of surface water and to prevent the ponding of water.
- The ground surface within the construction area should be sealed by a smooth-drum vibratory roller, or equivalent, and under no circumstances should be left uncompacted and exposed to moisture. Soils that become too wet for compaction should be removed and replaced with clean granular materials.
- Excavation and placement of fill should be observed by Aspect, the geotechnical engineer, to verify that all unsuitable materials are removed, and suitable compaction and Site drainage is achieved.
- Appropriate erosion and sedimentation BMPs should be strategically implemented in accordance with Washington State Department of Ecology and WSDOT recommendations.

6 Recommendations for Continuing Geotechnical Services

Throughout this report, we have provided recommendations where we consider it would be appropriate for Aspect to provide additional geotechnical input to the design and construction process. Additional recommendations are summarized in this section.

6.1 Additional Design and Consulting Services

Before construction begins, we recommend that Aspect:

- Continue to meet with the design team, as needed, to address geotechnical questions that may arise throughout the remainder of the design process.
- Review the design concepts as the design progresses to verify the geotechnical feasibility of site grading, retaining walls, and foundation systems and evaluate global stability as required. This may require additional explorations, depending on the design.
- Review the geotechnical elements of the project plans to see that the geotechnical engineering recommendations are properly interpreted.
- Provide an Environmentally Critical Area Impacts Statement of Risk with a final design report as required for City permitting.

6.2 Additional Construction Services

We are available to provide geotechnical engineering and monitoring services during construction. The integrity of the geotechnical elements depends on proper Site preparation and construction procedures. In addition, engineering decisions may have to be made in the field if variations in subsurface conditions become apparent.

During the construction phase of the Project, we recommend that Aspect be retained to perform the following tasks:

- Review applicable submittals
- Observe and evaluate subgrade preparation, structural fill placement, wall construction, and deep foundation installation
- Attend meetings, as needed
- Address other geotechnical engineering considerations that may arise during construction

The purpose of our observations is to verify compliance with design concepts and recommendations, and to allow design changes or evaluation of appropriate construction methods if subsurface conditions differ from those anticipated prior to the start of construction.

References

American Society of Civil Engineers (ASCE), 2018, ASCE 7 Hazard Tool, https://asce7hazardtool.online/, accessed February 2021.

Atwater, B.F., S. Musumi-Rokkaku, D. Satake, Y. Tsuji, K. Ueda, and D.K. Yamaguci (Atwater et al.), 2015, The orphan tsunami of 1700—Japanese clues to a parent earthquake in North America, U.S. Geological Survey, Professional Paper 1707.

Gray, D.H. and Leiser, A.T., 1982, Biotechnical Slope Protection and Erosion Control, Van Nostrand Reinhold: New York, N.Y.

International Code Council (ICC), 2018, International Building Code (IBC), Prepared by International Code Council, January 2018.

King County, 2021, iMap, https://gismaps.kingcounty.gov/iMap/, accessed February 20, 2021.

LPD Engineering, PLLC, Mercer Island House Permit Set, Issued August 27, 2021, MH Proj. No. A20.0085.00.

Mercer Island City Code, 2021, A Codification of the General Ordinances of the City of Mercer Island, Washington, Code Publishing Company, Seattle, Washington, https://mercerisland.municipal.codes/MICC accessed January 18, 2021.

PCS Structural Solutions, Mercer Island House Permit Set, Issued August 11, 2021, MH Proj. No. A20.0085.00.

Pratt, T.L., K.G. Troost, J.K. Odum, and W.J. Stephenson (Pratt et al.), 2015, Kinematics of shallow backthrusts in the Seattle fault zone, Washington State, Geosphere, v. 11, no. 6, p. 1–27, doi:10.1130/GES01179.1.

Terrane Surveying, 2021. Topographic and Boundary Survey for Buttenwieser/ Wiley Residence. 6838 96th Avenue SE, Mercer Island, WA 98040, February 4, 2021.

Troost, K.G. and Wisher, A.P., 2006, Geologic Map of Mercer Island, Washington, GeoMapNW, October 2006.

U.S. Geological Survey (USGS), 2016, Quaternary Fault and Fold Database of the United States, Seattle fault zone (Class A) No. 570, https://earthquake.usgs.gov/cfusion/qfault/show_report_AB.cfm?fault_id=570&section_id=, last review date November 25, 2016

Washington State Department of Natural Resources (DNR), 2004, Liquefaction Susceptibility and Site Class Maps of Washington State, By County, Washington Division of Geology and Earth Resources Open File Report 2004-20, by Palmer, S.P., S.L. Magsino, E.L. Bilderback, J.L. Poelstra, D.S. Folger, and R.A. Niggemann, 2004, September 2004.

Washington State Department of Transportation (WSDOT), 2021, Standard Specifications for Road, Bridge and Municipal Construction, Document M 41-10.

Limitations

Work for this project was performed for Janet Buttenwieser (Client), and this report was prepared consistent with recognized standards of professionals in the same locality and involving similar conditions, at the time the work was performed. No other warranty, expressed or implied, is made by Aspect Consulting, LLC (Aspect).

Recommendations presented herein are based on our interpretation of site conditions, geotechnical engineering calculations, and judgment in accordance with our mutually agreed-upon scope of work. Our recommendations are unique and specific to the project, site, and Client. Application of this report for any purpose other than the project should be done only after consultation with Aspect.

Variations may exist between the soil and groundwater conditions reported and those actually underlying the site. The nature and extent of such soil variations may change over time and may not be evident before construction begins. If any soil conditions are encountered at the site that are different from those described in this report, Aspect should be notified immediately to review the applicability of our recommendations.

Risks are inherent with any site involving slopes and no recommendations, geologic analysis, or engineering design can assure slope stability. Our observations, findings, and opinions are a means to identify and reduce the inherent risks to the Client.

It is the Client's responsibility to see that all parties to this project, including the designer, contractor, subcontractors, and agents, are made aware of this report in its entirety. At the time of this report, design plans and construction methods have not been finalized, and the recommendations presented herein are based on preliminary project information. If project developments result in changes from the preliminary project information, Aspect should be contacted to determine if our recommendations contained in this report should be revised and/or expanded upon.

The scope of work does not include services related to construction safety precautions. Site safety is typically the responsibility of the contractor, and our recommendations are not intended to direct the contractor’s site safety methods, techniques, sequences, or procedures. The scope of our work also does not include the assessment of environmental characteristics, particularly those involving potentially hazardous substances in soil or groundwater.

All reports prepared by Aspect for the Client apply only to the services described in the Agreement(s) with the Client. Any use or reuse by any party other than the Client is at the sole risk of that party, and without liability to Aspect. Aspect’s original files/reports shall govern in the event of any dispute regarding the content of electronic documents furnished to others.

Please refer to Appendix D titled “Report Limitations and Guidelines for Use” for additional information governing the use of this report.

We appreciate the opportunity to perform these services. If you have any questions please call Chip Barnett at 206.413.5398.

FIGURES

GIS Path: G:\projects\ButtenwieserResidence_200631\Delivered\01 Site Location Map.mxd || Coordinate System: NAD 1983 StatePlane Washington North FIPS 4601 Feet || Date Saved: 3/4/2021 || User: smortensen || Print Date: 3/4/2021

Site Location Map

Geotechnical Engineering Report
Buttenwieser/Wiley Residence
6838 96th Avenue SE
Mercer Island, Washington

Aspect CONSULTING	MAR-2021	BY: MR / SBM	FIGURE NO. 1
	PROJECT NO. 200631	REVISED BY: - - -	

Basemap Layer Credits || Esri, HERE, Garmin, (c) OpenStreetMap contributors, and the GIS user community
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⊕ Explorations

Contour - 10' Interval

Contour - 2' Interval

Site Parcel

Mercer Island Geologic Hazard Area

Seismic

Steep Slope

Potential Slide

Erosion

Note: Topographic Contours were obtained using survey data completed by Terrane Land Surveying and reference the North American Vertical Datum of 1988.

0 40 80
Feet

Site and Exploration Map

Geotechnical Engineering Report
Buttenwieser/Wiley Residence
6838 96th Avenue SE
Mercer Island, Washington

Aspect CONSULTING

SEP-2021	BY: MR / SBM	FIGURE NO.
PROJECT NO. 200631	REVISED BY: ETB / WEG	2

Basemap Layer Credits || EagleView Technologies, Inc.

GIS Path: G:\projects\ButtenwieserResidence_200631\Delivered\02 Site and Exploration Map.mxd || Coordinate System: NAD 1983 StatePlane Washington North FIPS 4601 Feet || Date Saved: 9/1/2021 || User: bgrimm || Print Date: 9/1/2021

APPENDIX A

Subsurface Exploration Logs

A. Subsurface Exploration Logs

On February 1 and 2, 2021, Aspect Consulting, LLC (Aspect) completed six machine-drilled borings (designated AB-01 through AB-06) at the Site. The machine-drilled borings were advanced with hollow-stem auger drilling methods using a portable tracked drill rig operated by Geologic Drilling Partners, Inc. under subcontract to Aspect.

Disturbed soil samples were obtained at 2.5- or 5-foot intervals using the Standard Penetration Test (SPT) in accordance with ASTM D1586, *Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils* (ASTM, 2018). Typically, the Standard Penetration Test involves driving a 2-inch-outside-diameter split-barrel sampler a distance of 18 inches into the soil with a 140-pound hammer free-falling a distance of 30 inches (the drill rig employed on this project used rope and cathead to raise and lower the hammer). The number of blows for each 6-inch interval is recorded and the number of blows required to drive the sampler for the final two intervals (a total of 12 inches) is known as the Standard Penetration Resistance (“N-value”) or blow count. The N-value provides a measure of relative density of granular soils or the relative consistency of cohesive soils. Upon completion, the machine-drilled borings were backfilled with 3/8-inch bentonite chips in accordance with requirements of the Washington State Department of Ecology.

An Aspect engineer or geologist was present throughout the exploration program to observe the drilling procedures, assist in sampling, and to prepare descriptive logs of the explorations. Soils were identified in general accordance with ASTM D2488, *Standard Practice for Description and Identification of Soils* (Visual-Manual Procedure). The summary exploration logs represent our interpretation of the contents of the field logs. The stratigraphic contacts shown on the individual summary logs represent the approximate boundaries between soil types; actual transitions may be more gradual. The subsurface conditions depicted are only for the specific date and locations reported, and therefore, are not necessarily representative of other locations and times.

Coarse-Grained Soils - More than 50%1 Retained on No. 200 Sieve	Gravels - More than 50%^{1} of Coarse Fraction Retained on No. 4 Sieve	≤5% Fines	▨	**GW**	Well-graded GRAVEL Well-graded GRAVEL WITH SAND
			▨	**GP**	Poorly-graded GRAVEL Poorly-graded GRAVEL WITH SAND
		≥15% Fines	▨	**GM**	SILTY GRAVEL SILTY GRAVEL WITH SAND
			▨	**GC**	CLAYEY GRAVEL CLAYEY GRAVEL WITH SAND
	Sands - 50%^{1} or More of Coarse Fraction Passes No. 4 Sieve	≤5% Fines	▨	**SW**	Well-graded SAND Well-graded SAND WITH GRAVEL
			▨	**SP**	Poorly-graded SAND Poorly-graded SAND WITH GRAVEL
		≥15% Fines	▨	**SM**	SILTY SAND SILTY SAND WITH GRAVEL
			▨	**SC**	CLAYEY SAND CLAYEY SAND WITH GRAVEL
Fine-Grained Soils - 50%1 or More Passes No. 200 Sieve	Silts and Clays Liquid Limit Less than 50%		▨	**ML**	SILT SANDY or GRAVELLY SILT SILT WITH SAND SILT WITH GRAVEL
			▨	**CL**	LEAN CLAY SANDY or GRAVELLY LEAN CLAY LEAN CLAY WITH SAND LEAN CLAY WITH GRAVEL
			▨	**OL**	ORGANIC SILT SANDY or GRAVELLY ORGANIC SILT ORGANIC SILT WITH SAND ORGANIC SILT WITH GRAVEL
	Silts and Clays Liquid Limit 50% or More		▨	**MH**	ELASTIC SILT SANDY or GRAVELLY ELASTIC SILT ELASTIC SILT WITH SAND ELASTIC SILT WITH GRAVEL
			▨	**CH**	FAT CLAY SANDY or GRAVELLY FAT CLAY FAT CLAY WITH SAND FAT CLAY WITH GRAVEL
			▨	**OH**	ORGANIC CLAY SANDY or GRAVELLY ORGANIC CLAY ORGANIC CLAY WITH SAND ORGANIC CLAY WITH GRAVEL
Highly Organic Soils			▨	**PT**	PEAT and other mostly organic soils

"WITH SILT" or "WITH CLAY" means 5 to 15% silt and clay, denoted by a "-" in the group name; e.g., SP-SM • "SILTY" or "CLAYEY" means >15% silt and clay • "WITH SAND" or "WITH GRAVEL" means 15 to 30% sand and gravel. • "SANDY" or "GRAVELLY" means >30% sand and gravel. • "Well-graded" means approximately equal amounts of fine to coarse grain sizes • "Poorly graded" means unequal amounts of grain sizes • Group names separated by "/" means soil contains layers of the two soil types; e.g., SM/ML.

Soils were described and identified in the field in general accordance with the methods described in ASTM D2488. Where indicated in the log, soils were classified using ASTM D2487 or other laboratory tests as appropriate. Refer to the report accompanying these exploration logs for details.

1. Estimated or measured percentage by dry weight
2. (SPT) Standard Penetration Test (ASTM D1586)
3. Determined by SPT, DCPT (ASTM STP399) or other field methods. See report text for details.

GEOTECHNICAL LAB TESTS

MC = Natural Moisture Content
PS = Particle Size Distribution
FC = Fines Content (% < 0.075 mm)
GH = Hydrometer Test
AL = Atterberg Limits
C = Consolidation Test
Str = Strength Test
OC = Organic Content (% Loss by Ignition)
Comp = Proctor Test
K = Hydraulic Conductivity Test
SG = Specific Gravity Test

CHEMICAL LAB TESTS

Organic Chemicals

BTEX = Benzene, Toluene, Ethylbenzene, Xylenes
TPH-Dx = Diesel and Oil-Range Petroleum Hydrocarbons
TPH-G = Gasoline-Range Petroleum Hydrocarbons
VOCs = Volatile Organic Compounds
SVOCs = Semi-Volatile Organic Compounds
PAHs = Polycyclic Aromatic Hydrocarbon Compounds
PCBs = Polychlorinated Biphenyls

Metals

RCRA8 = As, Ba, Cd, Cr, Pb, Hg, Se, Ag, (d = dissolved, t = total)
MTCA5 = As, Cd, Cr, Hg, Pb (d = dissolved, t = total)
PP-13 = Ag, As, Be, Cd, Cr, Cu, Hg, Ni, Pb, Sb, Se, Tl, Zn (d=dissolved, t=total)

FIELD TESTS

PID = Photoionization Detector
Sheen = Oil Sheen Test
SPT^2 = Standard Penetration Test
NSPT = Non-Standard Penetration Test
DCPT = Dynamic Cone Penetration Test

COMPONENT DEFINITIONS

Descriptive Term		***Size Range and Sieve Number***
Boulders	=	Larger than 12 inches
Cobbles	=	3 inches to 12 inches
Coarse Gravel	=	3 inches to 3/4 inches
Fine Gravel	=	3/4 inches to No. 4 (4.75 mm)
Coarse Sand	=	No. 4 (4.75 mm) to No. 10 (2.00 mm)
Medium Sand	=	No. 10 (2.00 mm) to No. 40 (0.425 mm)
Fine Sand	=	No. 40 (0.425 mm) to No. 200 (0.075 mm)
Silt and Clay	=	Smaller than No. 200 (0.075 mm)

ESTIMATED^1 PERCENTAGE

% by Weight		***Modifier***	***% by Weight***		***Modifier***
<1	=	Subtrace	15 to 25	=	Little
1 to <5	=	Trace	30 to 45	=	Some
5 to 10	=	Few	>50	=	Mostly

MOISTURE CONTENT

Dry = Absence of moisture, dusty, dry to the touch
Slightly Moist = Perceptible moisture
Moist = Damp but no visible water
Very Moist = Water visible but not free draining
Wet = Visible free water, usually from below water table

RELATIVE DENSITY

Non-Cohesive or Coarse-Grained Soils

Density³		***SPT² Blows/Foot***	***Penetration with 1/2" Diameter Rod***
Very Loose	=	0 to 4	≥ 2'
Loose	=	5 to 10	1' to 2'
Medium Dense	=	11 to 30	3" to 1'
Dense	=	31 to 50	1" to 3"
Very Dense	=	> 50	< 1"

CONSISTENCY

Cohesive or Fine-Grained Soils

Consistency³		***SPT² Blows/Foot***	***Manual Test***
Very Soft	=	0 to 1	Penetrated >1" easily by thumb. Extrudes between thumb & fingers.
Soft	=	2 to 4	Penetrated 1/4" to 1" easily by thumb. Easily molded.
Medium Stiff	=	5 to 8	Penetrated >1/4" with effort by thumb. Molded with strong pressure.
Stiff	=	9 to 15	Indented ~1/4" with effort by thumb.
Very Stiff	=	16 to 30	Indented easily by thumbnail.
Hard	=	> 30	Indented with difficulty by thumbnail.

GEOLOGIC CONTACTS

Observed and Distinct	Observed and Gradual	Inferred
———	———	— — — —

Aspect CONSULTING

Exploration Log Key

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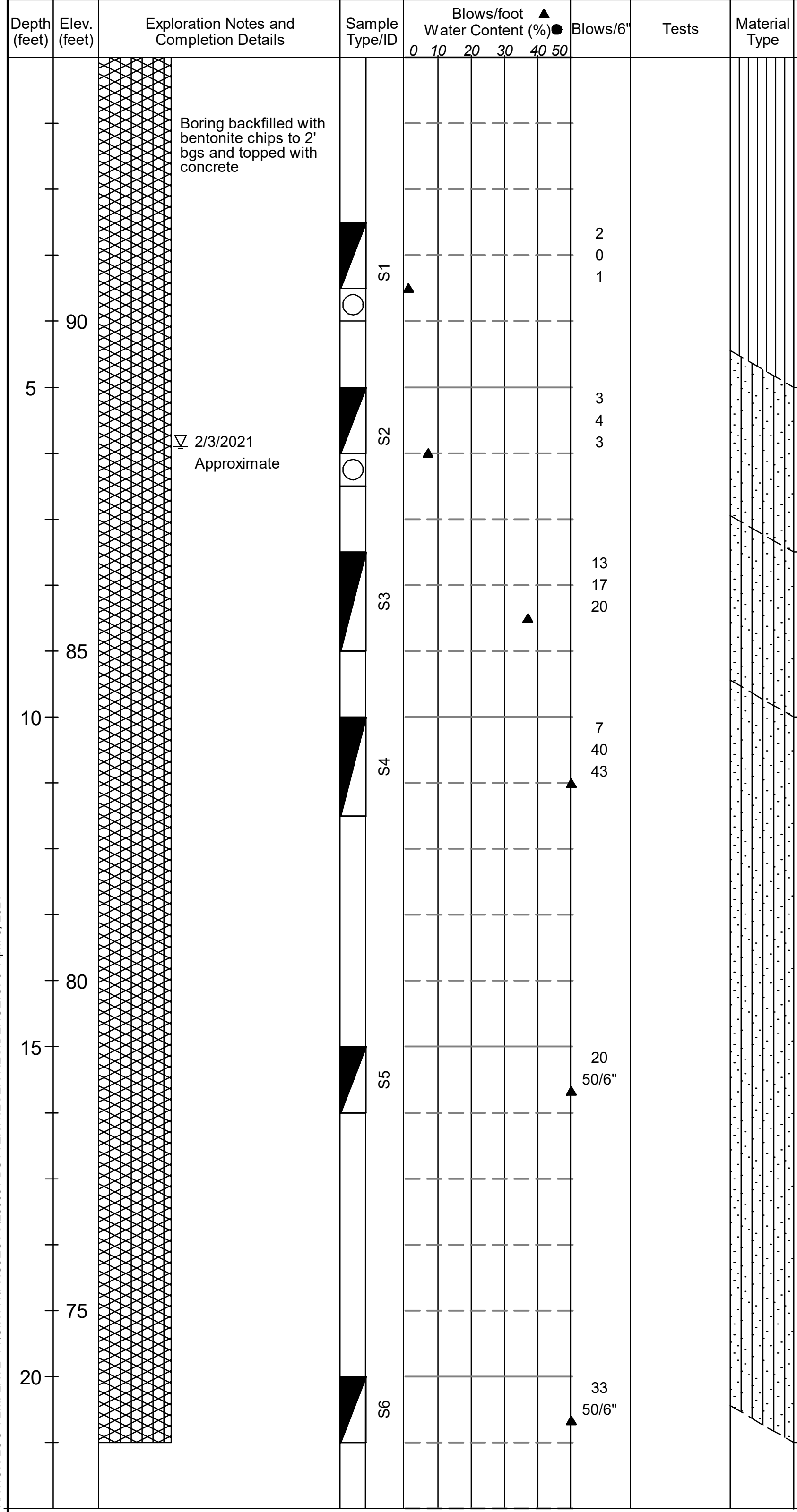
Aspect CONSULTING

Buttenwieser/Wiley Residence - 200631

Geotechnical Exploration Log

Project Address & Site Specific Location			*Coordinates (Lat,Lon WGS84)*	*Exploration Number*
6838 96th Ave SE, Mercer Island, WA, Gravel Turnout at Top of Drive			47.5412, -122.2110 (est)	**AB-01**
Contractor	*Equipment*	*Sampling Method*	*Ground Surface Elev. (NAVD88)*	
Geologic Drill Partners, Inc.	Mini Track Rig	Rope & cathead; 140 lb hammer; 30" drop	94' (est)	
Operator	*Exploration Method(s)*	*Work Start/Completion Dates*	*Top of Casing Elev. (NAVD88)*	*Depth to Water (Below GS)*
Jeremy	6" OD X 2.25" ID Hollow Stem Auger	2/3/2021	NA	5.9' (ATD)



Boring backfilled with bentonite chips to 2' bgs and topped with concrete

∇ 2/3/2021
Approximate

Description

Fill

SANDY SILT (ML); very soft, wet, gray; nonplastic; some roots and woody debris

5

SILTY SAND (SM); loose, wet, gray to brown; fine sand

Weathered Pre-Olympia Nonglacial Deposits

SILTY SAND (SM); dense, wet, light brown to red brown with red mottling; fine to coarse sand; trace fine to coarse, rounded gravel

10

Coarse-Grained Pre-Olympia Nonglacial Deposits

SILTY SAND WITH GRAVEL (SM); very dense, slight moist, gray with multicolored clasts; fine to coarse sand; fine to coarse, subrounded to subangular gravel; partings of silt and clay; diamict texture

15

Becomes increasingly sandy with fine, subrounded gravel

20

Diamict texture becomes more pronounced

Bottom of exploration at 21 ft. bgs.

Legend

Plastic Limit ├──┤ Liquid Limit

Sample Type

○ No Soil Sample Recovery

◩ Split Barrel 2" X 1.375" (SPT)

Water Level

∇ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: M. Reiter
Approved by: C. Barnett

Exploration Log AB-01

Sheet 1 of 1

NEW STANDARD EXPLORATION LOG TEMPLATE P:\GINTW\PROJECTS\200631 BUTTENWIESER RESIDENCE.GPJ April 6, 2021

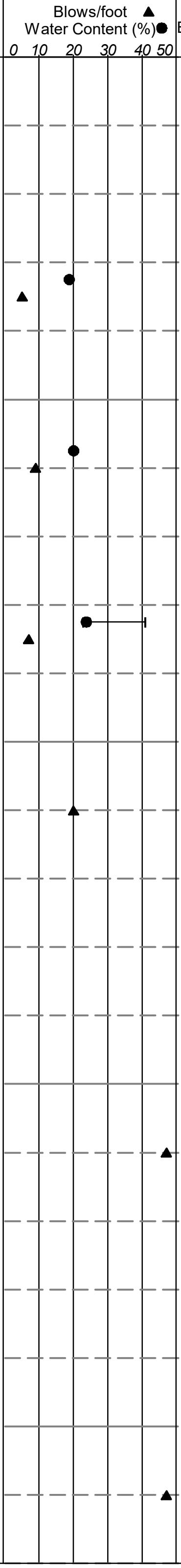
Aspect CONSULTING

Buttenwieser/Wiley Residence - 200631

Geotechnical Exploration Log

Project Address & Site Specific Location			*Coordinates (Lat,Lon WGS84)*	*Exploration Number*
6838 96th Ave SE, Mercer Island, WA, Conc. Driveway			47.5411, -122.2106 (est)	**AB-02**
Contractor	*Equipment*	*Sampling Method*	*Ground Surface Elev. (NAVD88)*	
Geologic Drill Partners, Inc.	Mini Track Rig	Rope & cathead; 140 lb hammer; 30" drop	76' (est)	
Operator	*Exploration Method(s)*	*Work Start/Completion Dates*	*Top of Casing Elev. (NAVD88)*	*Depth to Water (Below GS)*
Jeremy	6" OD X 2.25" ID Hollow Stem Auger	2/2/2021	NA	No Water Encountered

Depth (feet)	Elev. (feet)	Exploration Notes and Completion Details	Sample Type/ID	Blows/6"	Tests	Description	Depth (ft)
						Pavement CONCRETE; 4 inches concrete pavement with intermittent rebar	
	75	Boring backfilled with bentonite chips				**Fill** SANDY SILT (ML); medium stiff, very moist, red brown to gray; nonplastic; fine sand	
			S1	2 2 3	MC		
5			S2	4 5 4	MC	Becomes stiff	5
	70						
			S3	4 3 4	AL, MC LL=41% PL=23%	CLAY (CL); medium stiff, moist, light brown with red mottling; medium plasticity, medium toughness, no dilatancy, medium dry strength; trace fine, rounded gravel	
10			S4	3 10 10		**Weathered Pre-Olympia Nonglacial Deposits** SILTY SAND (SM); medium dense, very moist, brown to gray; fine to medium sand; trace organics	10
	65						
15			S5	13 23 24		**Coarse-Grained Pre-Olympia Nonglacial Deposits** SAND WITH SILT (SP-SM); dense, moist, gray brown; fine sand	15
	60						
20			S6	19 21 26		SILTY SAND (SM); dense, wet, gray brown; fine to medium sand	20
	55						
						Bottom of exploration at 21.5 ft. bgs.	



Legend

Sample Type: ◩ Split Barrel 2" X 1.375" (SPT)

Plastic Limit ├─┤ Liquid Limit

Water Level: No Water Encountered

See Exploration Log Key for explanation of symbols

Logged by: M. Reiter
Approved by: C. Barnett

Exploration Log AB-02

Sheet 1 of 1

Aspect CONSULTING

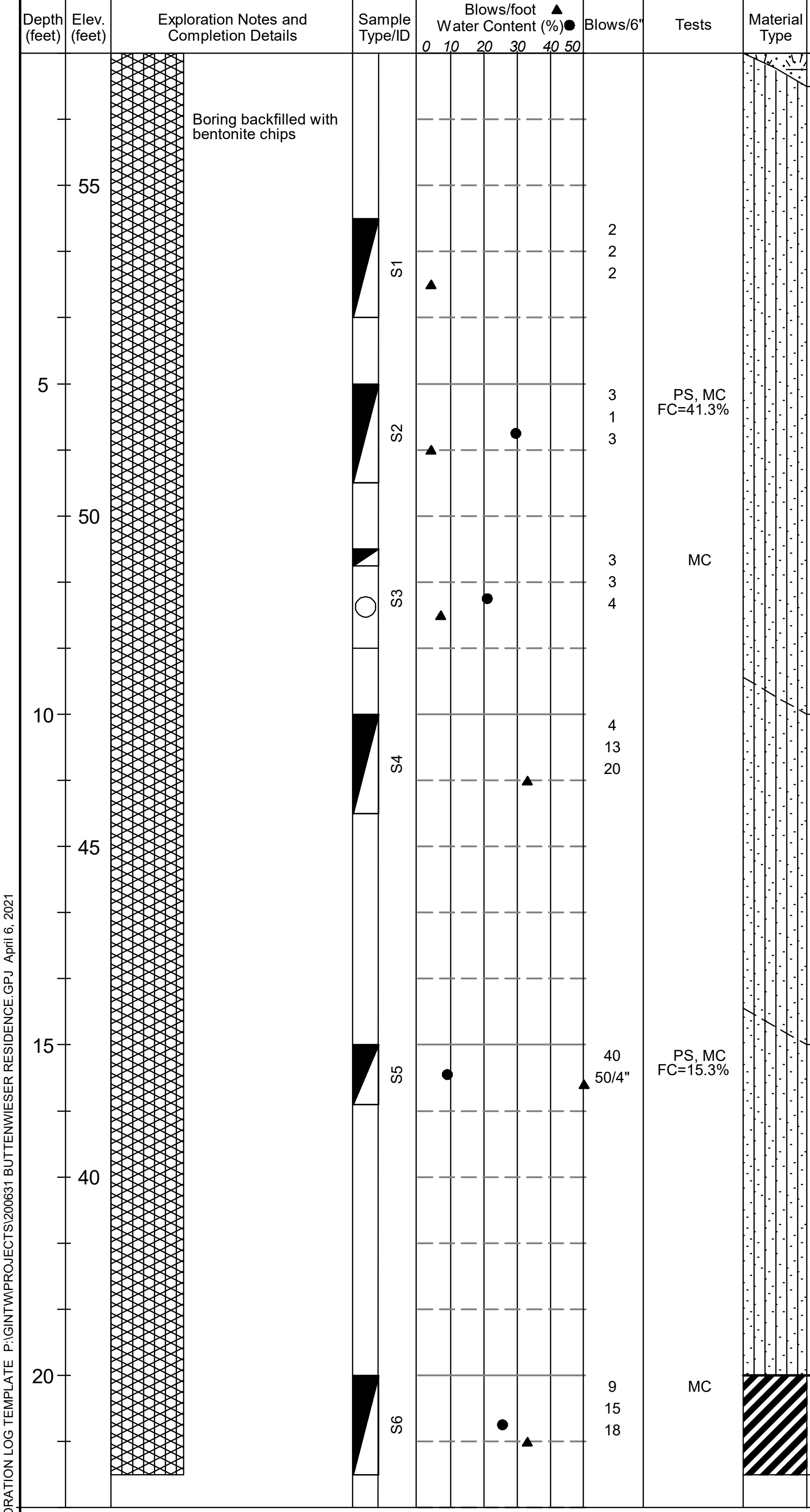
Buttenwieser/Wiley Residence - 200631

Geotechnical Exploration Log

Project Address & Site Specific Location	*Coordinates (Lat,Lon WGS84)*	*Exploration Number*
6838 96th Ave SE, Mercer Island, WA, Planters Behind Garage	47.5410, -122.2105 (est)	**AB-03**

Contractor	*Equipment*	*Sampling Method*	*Ground Surface Elev. (NAVD88)*
Geologic Drill Partners, Inc.	Mini Track Rig	Rope & cathead; 140 lb hammer; 30" drop	57' (est)

Operator	*Exploration Method(s)*	*Work Start/Completion Dates*	*Top of Casing Elev. (NAVD88)*	*Depth to Water (Below GS)*
Jeremy	6" OD X 2.25" ID Hollow Stem Auger	2/2/2021	NA	No Water Encountered



Sample Type/ID	Blows/6"	Tests
S1	2 2 2	
S2	3 1 3	PS, MC FC=41.3%
S3	3 3 4	MC
S4	4 13 20	
S5	40 50/4"	PS, MC FC=15.3%
S6	9 15 18	MC

Description

Topsoil

TOPSOIL; very loose, moist, dark brown; mostly silt with sand; abundant organics

Fill

SILTY SAND (SM); very loose, very moist, gray brown; fine to coarse sand; partings of nonplastic silt

Becomes light gray to red brown

Becomes loose, moist, gray

Weathered Pre-Olympia Nonglacial Deposits (10)

SILTY SAND WITH GRAVEL (SM); medium dense, wet, light brown to gray; fine to coarse sand; fine, subrounded to subangular gravel; trace black organics

Coarse-Grained Pre-Olympia Nonglacial Deposits (15)

SILTY SAND WITH GRAVEL (SM); very dense, very moist, gray brown; fine to coarse sand; fine, subrounded to subangular gravel

Fine-Grained Pre-Olympia Nonglacial Deposits (20)

CLAY (CH); hard, slightly moist, blue gray; medium to high plasticity, medium toughness, no dilatancy, high to very high dry strength

Bottom of exploration at 21.5 ft. bgs.

Legend

Sample Type

○ No Soil Sample Recovery

◪ Split Barrel 2" X 1.375" (SPT)

Plastic Limit ├──┤ Liquid Limit

Water Level

No Water Encountered

See Exploration Log Key for explanation of symbols

Logged by: M. Reiter
Approved by: C. Barnett

Exploration Log AB-03

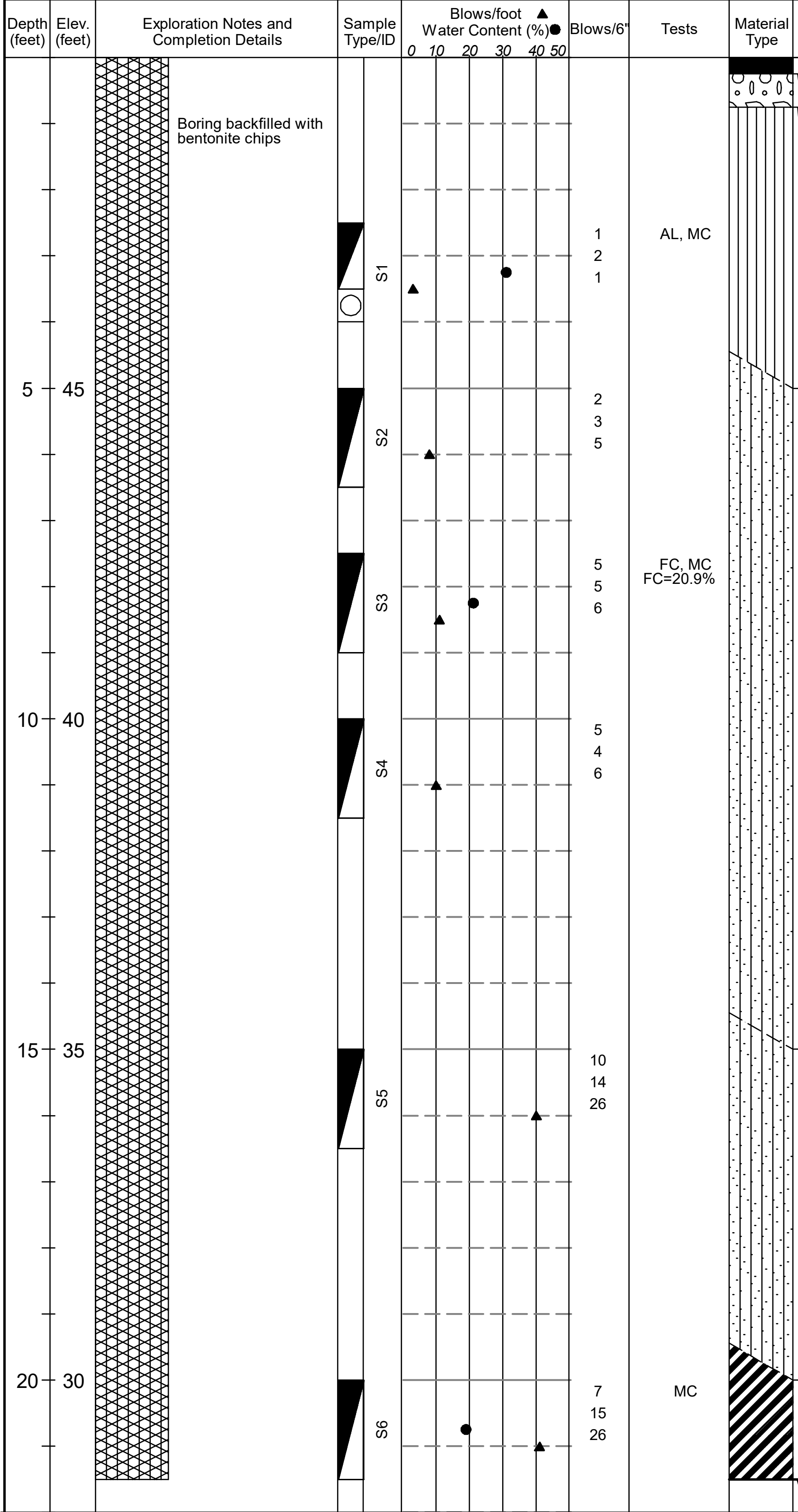
Sheet 1 of 1

Aspect CONSULTING

Buttenwieser/Wiley Residence - 200631

Geotechnical Exploration Log

Project Address & Site Specific Location 6838 96th Ave SE, Mercer Island, WA, Top of Driveway Rockery			*Coordinates (Lat,Lon WGS84)* 47.5410, -122.2101 (est)	*Exploration Number* **AB-04**
Contractor Geologic Drill Partners, Inc.	*Equipment* Mini Track Rig	*Sampling Method* Rope & cathead; 140 lb hammer; 30" drop	*Ground Surface Elev. (NAVD88)* 50' (est)	
Operator Jeremy	*Exploration Method(s)* 6" OD X 2.25" ID Hollow Stem Auger	*Work Start/Completion Dates* 2/2/2021	*Top of Casing Elev. (NAVD88)* NA	*Depth to Water (Below GS)* No Water Encountered



S1	1 2 1	AL, MC
S2	2 3 5	
S3	5 5 6	FC, MC FC=20.9%
S4	5 4 6	
S5	10 14 26	
S6	7 15 26	MC

Description

Pavement

ASPHALT; 3 inches hot mix asphalt

GRAVEL WITH SAND (GW); 6 inches crushed aggregate surfacing

Fill

SILT WITH SAND (ML); soft, moist, dark gray; nonplastic; fine sand; trace woody debris

SILTY SAND (SM); loose, moist, gray with red mottling; fine sand; trace organics

Becomes medium dense; fine to coarse sand

Becomes loose, wet; fine sand

Coarse-Grained Pre-Olympia Nonglacial Deposits

SILTY SAND WITH GRAVEL (SM); dense, slightly moist, brown; fine to coarse sand; fine, rounded gravel; diamict texture; trace charcoal

Fine-Grained Pre-Olympia Nonglacial Deposits

CLAY (CH); hard, slightly moist, blue gray; medium to high plasticity, medium toughness, no dilatancy, high to very high dry strength; trace fine sand; trace fine, subrounded to subangular gravel

Legend

Sample Type
- ○ No Soil Sample Recovery
- ◩ Split Barrel 2" X 1.375" (SPT)

Plastic Limit ├──┤ Liquid Limit

Water Level: No Water Encountered

Bottom of exploration at 21.5 ft. bgs.
See Exploration Log Key for explanation of symbols

Logged by: M. Reiter
Approved by: C. Barnett

Exploration Log AB-04

Sheet 1 of 1

NEW STANDARD EXPLORATION LOG TEMPLATE P:\GINTW\PROJECTS\200631 BUTTENWIESER RESIDENCE.GPJ April 6, 2021

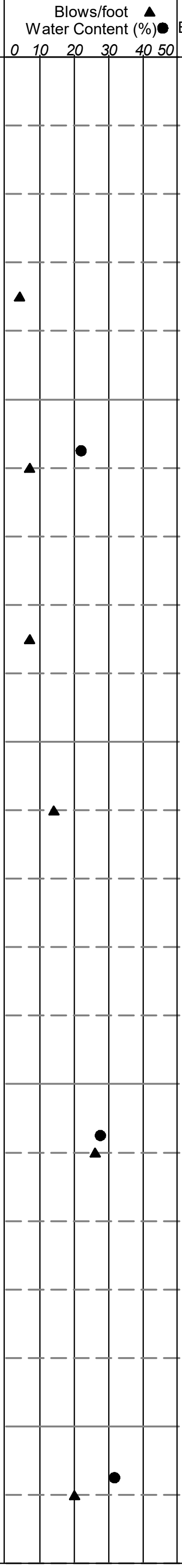
Aspect CONSULTING

Buttenwieser/Wiley Residence - 200631

Geotechnical Exploration Log

Project Address & Site Specific Location 6838 96th Ave SE, Mercer Island, WA, Bot. of Driveway Rockery		*Coordinates (Lat,Lon WGS84)* 47.5410, -122.2100 (est)	*Exploration Number* **AB-05**
Contractor Geologic Drill Partners, Inc.	*Equipment* Mini Track Rig	*Sampling Method* Rope & cathead; 140 lb hammer; 30" drop	*Ground Surface Elev. (NAVD88)* 42' (est)
Operator Jeremy	*Exploration Method(s)* 6" OD X 2.25" ID Hollow Stem Auger	*Work Start/Completion Dates* 2/3/2021	*Top of Casing Elev. (NAVD88)* NA
			Depth to Water (Below GS) No Water Encountered

Depth (feet)	Elev. (feet)	Exploration Notes and Completion Details	Sample Type/ID	Blows/6"	Tests	Description	Depth (ft)
		Boring backfilled with bentonite chips				**Topsoil** TOPSOIL; very loose, very moist, brown; mostly silt with fine sand; abundant roots and woody debris **Fill** SANDY SILT (ML); soft, very moist, gray; nonplastic; fine sand; some roots	
	40		S1	2 1 3			
5			S2	4 3 4	PS, MC FC=20.1%	SILTY SAND (SM); loose, very moist, gray; fine to medium sand	5
	35		S3	3 3 4		Grades to brown	
10			S4 ○	5 7 7		Becomes medium dense, light brown; fine to medium sand; trace fine, rounded gravel	10
	30						
15			S5	9 11 15	MC	**Fine-Grained Pre-Olympia Nonglacial Deposits** CLAY (CH); very stiff, slightly moist, blue gray; medium to high plasticity, medium toughness, no dilatancy, high to very high dry strength	15
	25					Drilling action suggests very stiff to hard soil	
20			S6	7 9 11	MC		20
						Bottom of exploration at 21.5 ft. bgs.	



Legend

Sample Type
- ○ No Soil Sample Recovery
- ◪ Split Barrel 2" X 1.375" (SPT)

Plastic Limit ├──┤ Liquid Limit

Water Level: No Water Encountered

See Exploration Log Key for explanation of symbols

Logged by: M. Reiter
Approved by: C. Barnett

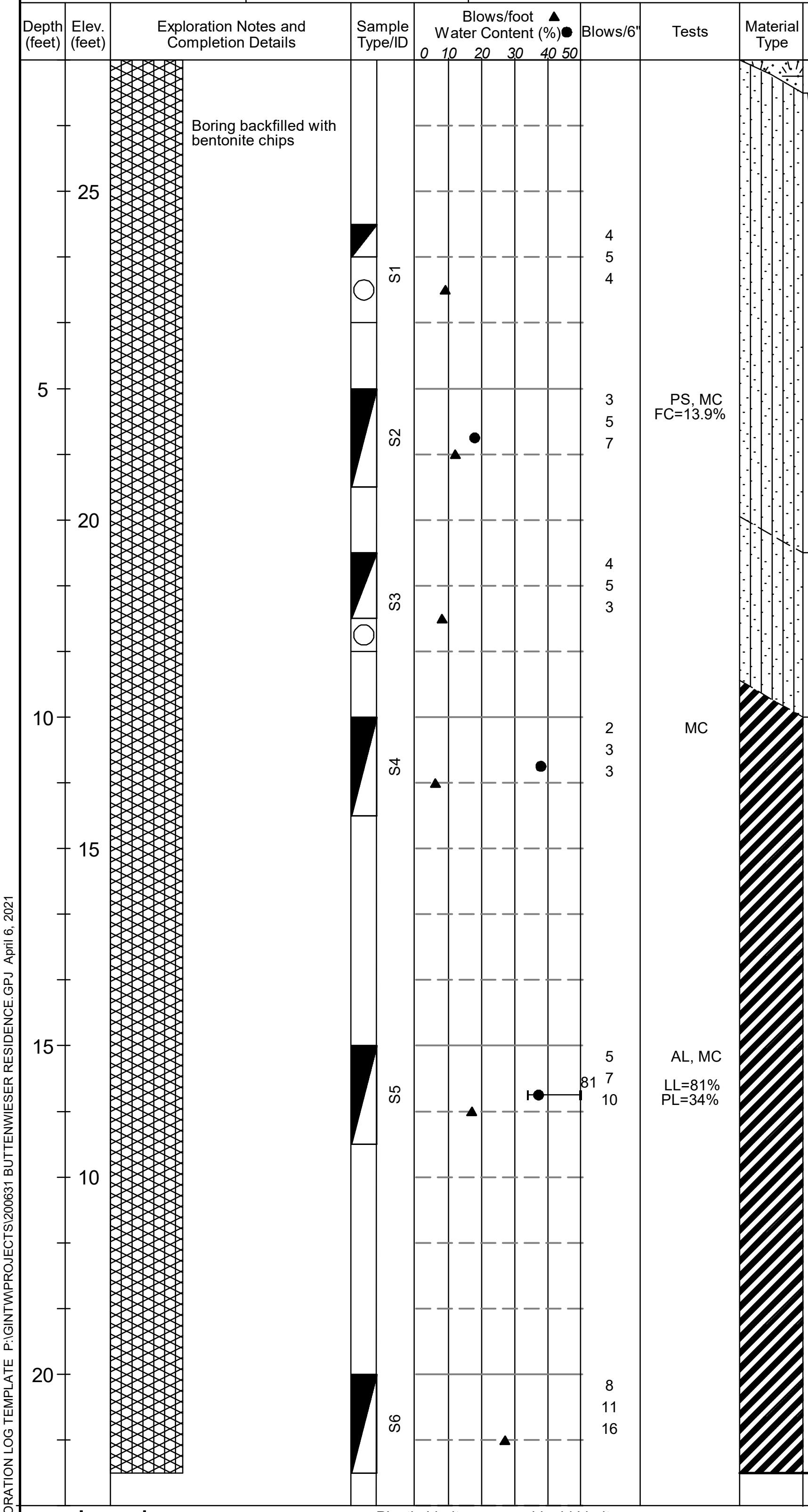
Exploration Log AB-05

Sheet 1 of 1

Buttenwieser/Wiley Residence - 200631

Geotechnical Exploration Log

Project Address & Site Specific Location	6838 96th Ave SE, Mercer Island, WA, S.E. House Corner	*Coordinates (Lat,Lon WGS84)*	47.5410, -122.2096 (est)	*Exploration Number* **AB-06**
Contractor	Geologic Drill Partners, Inc.	*Equipment*	Mini Track Rig	
Sampling Method	Rope & cathead; 140 lb hammer; 30" drop	*Ground Surface Elev. (NAVD88)*	27' (est)	
Operator	Jeremy	*Exploration Method(s)*	6" OD X 2.25" ID Hollow Stem Auger	
Work Start/Completion Dates	2/3/2021	*Top of Casing Elev. (NAVD88)*	NA	*Depth to Water (Below GS)* No Water Encountered



Sample Type/ID	Blows/6"	Tests
S1	4 5 4	
S2	3 5 7	PS, MC FC=13.9%
S3	4 5 3	
S4	2 3 3	MC
S5	5 7 10	AL, MC LL=81% PL=34%
S6	8 11 16	

Description

Topsoil

TOPSOIL; very loose, very moist, brown; mostly silt with fine sand; abundant organics and roots

Fill

SILTY SAND (SM); loose, very moist, gray; fine to medium sand

5 Becomes medium dense, dark gray; trace coarse sand; trace fine, subrounded gravel

Weathered Pre-Olympia Nonglacial Deposits

SILTY SAND (SM); loose, wet, brown with red mottling; fine to coarse sand; trace fine to coarse, rounded gravel; trace black organics

10 **Fine-Grained Pre-Olympia Nonglacial Deposits**

CLAY (CH); medium stiff, slightly moist, blue gray; medium to high plasticity, medium toughness, no dilatancy, high to very high dry strength

15 Becomes very stiff, slightly moist

20

Bottom of exploration at 21.5 ft. bgs.

Legend

Sample Type

○ No Soil Sample Recovery

◩ Split Barrel 2" X 1.375" (SPT)

Plastic Limit ⊢—⊣ Liquid Limit

Water Level

No Water Encountered

See Exploration Log Key for explanation of symbols

Logged by: M. Reiter
Approved by: C. Barnett

Exploration Log
AB-06

Sheet 1 of 1

APPENDIX B

Laboratory Testing Results

B. Laboratory Testing Results

Laboratory tests were conducted on selected soil samples to characterize certain engineering (physical) properties of the Site soils. Laboratory testing included determination of natural moisture content, fines content, Atterberg Limits, and grain-size distribution, in general accordance with appropriate ASTM test methods.

The moisture content of selected samples was analyzed in general accordance with ASTM D2216, *Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass*. The fines content of selected samples was analyzed in general accordance with ASTM D1140, Standard Test Methods of Determining the Amount of Material Finer than 75-mm (No. 200) Sieve in Soils by Washing. The grain-size distribution of selected samples was analyzed in general accordance with ASTM D6913, *Standard Test Method for Particle-Size Analysis of Soils without Hydrometer Determination of Fines Content*. The Atterberg Limits were analyzed in general accordance with ASTM D4318, *Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils*.

The results of the laboratory tests are presented in this appendix; moisture content and Atterberg Limit results are also presented graphically on the boring logs in Appendix A. The results of the grain-size distribution tests are presented as curves in this appendix, plotting percent finer by weight versus grain size.

9644 153rd Ave NE
Redmond, WA 98052
425-869-6750

Minus No. 200 Wash

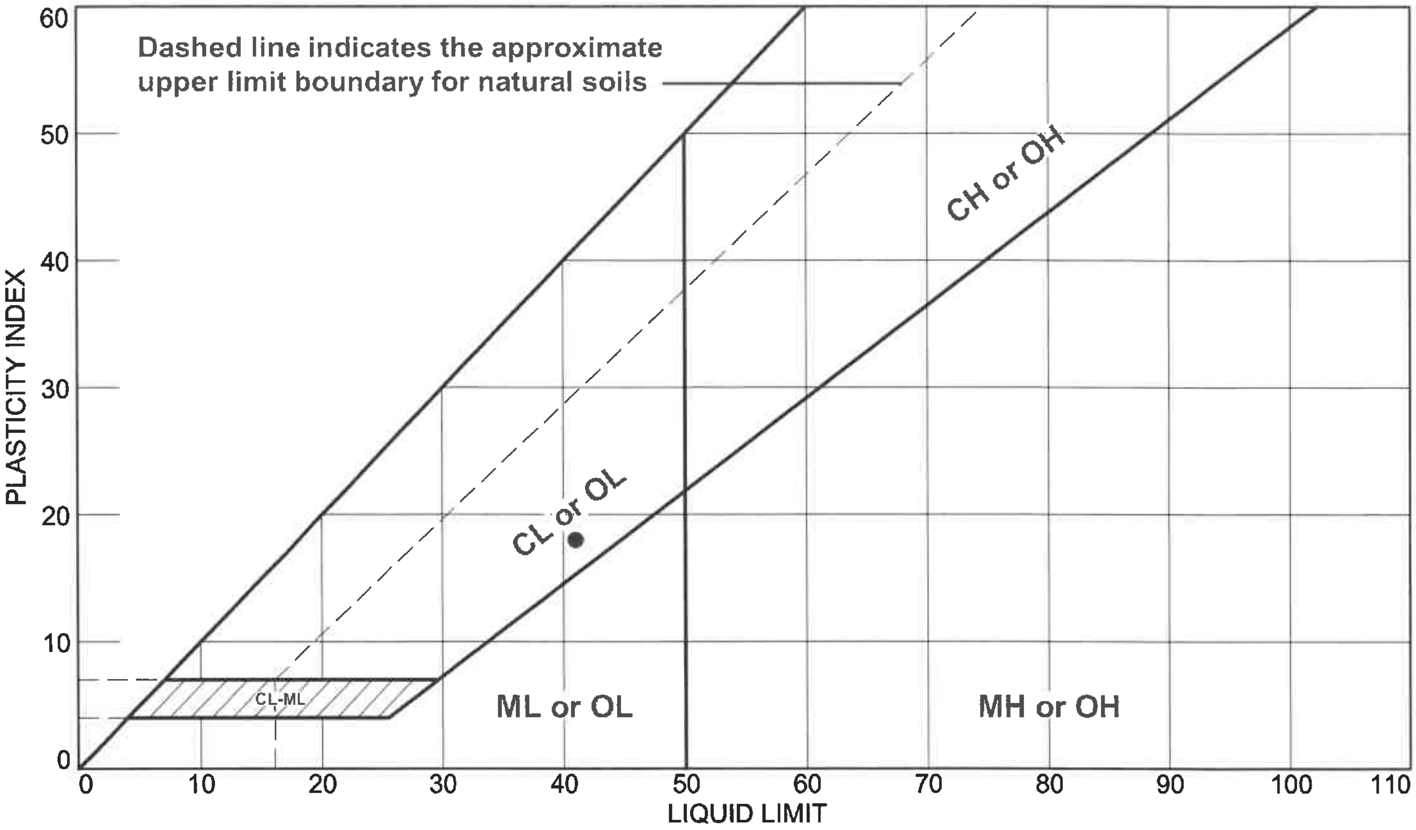
ASTM C117

Project Number: 08-175/200631
Project Name: Buttenweiser Residence
Lab Number: 8385

Technician: AD
Received: 2/5/2021
Start Date: 2/5/2021
Finish Date: 2/18/2021

HMA LAB NO	Boring No	Sample Number	Depth (ft)	Tare Weight (g)	Tare+Dry Weight Before Wash (g)	Tare+Dry Weight After Wash (g)	% Retained	% PASSING
8385-9	AB-04	S-3	7.5	15.9	358.3	286.6	79.1	20.9

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA

	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	LIQUIDITY INDEX	USCS
●	AB-02 / S-3	8385-3	7.5	24.0	23	41	18	0.1	CL

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Buttenweiser Residence

Project No.: 08-175/200631 **Figure**

Tested By: AD **Checked By:** JAM

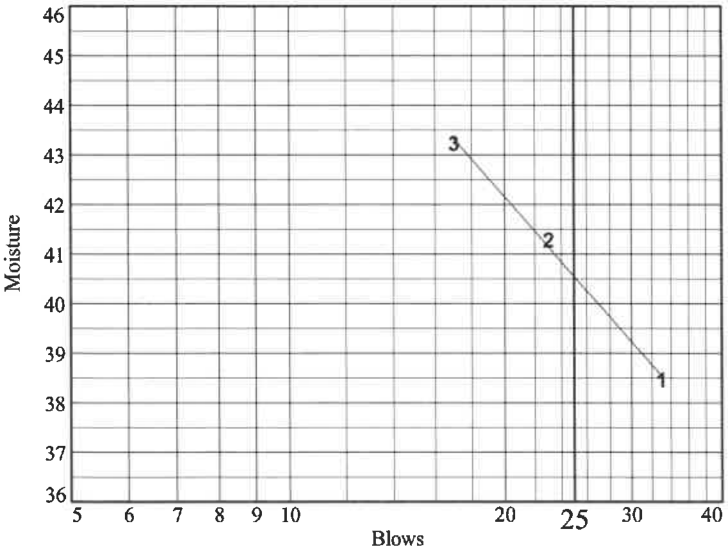
LIQUID AND PLASTIC LIMIT TEST DATA

2/18/2021

Client: Aspect Consulting
Project: Buttenweiser Residence
Project Number: 08-175/200631
Location: AB-02 / S-3
Depth: 7.5
Sample Number: 8385-3
Material Description: Lean CLAY with sand
USCS: CL
Tested by: AD
Checked by: JAM

Liquid Limit Data

Run No.	1	2	3	4	5	6
Wet+Tare	29.20	33.90	30.22			
Dry+Tare	24.86	28.02	25.26			
Tare	13.58	13.78	13.79			
# Blows	33	23	17			
Moisture	38.5	41.3	43.2			



Liquid Limit= 41
Plastic Limit= 23
Plasticity Index= 18
Natural Moisture= 24.0
Liquidity Index= 0.1

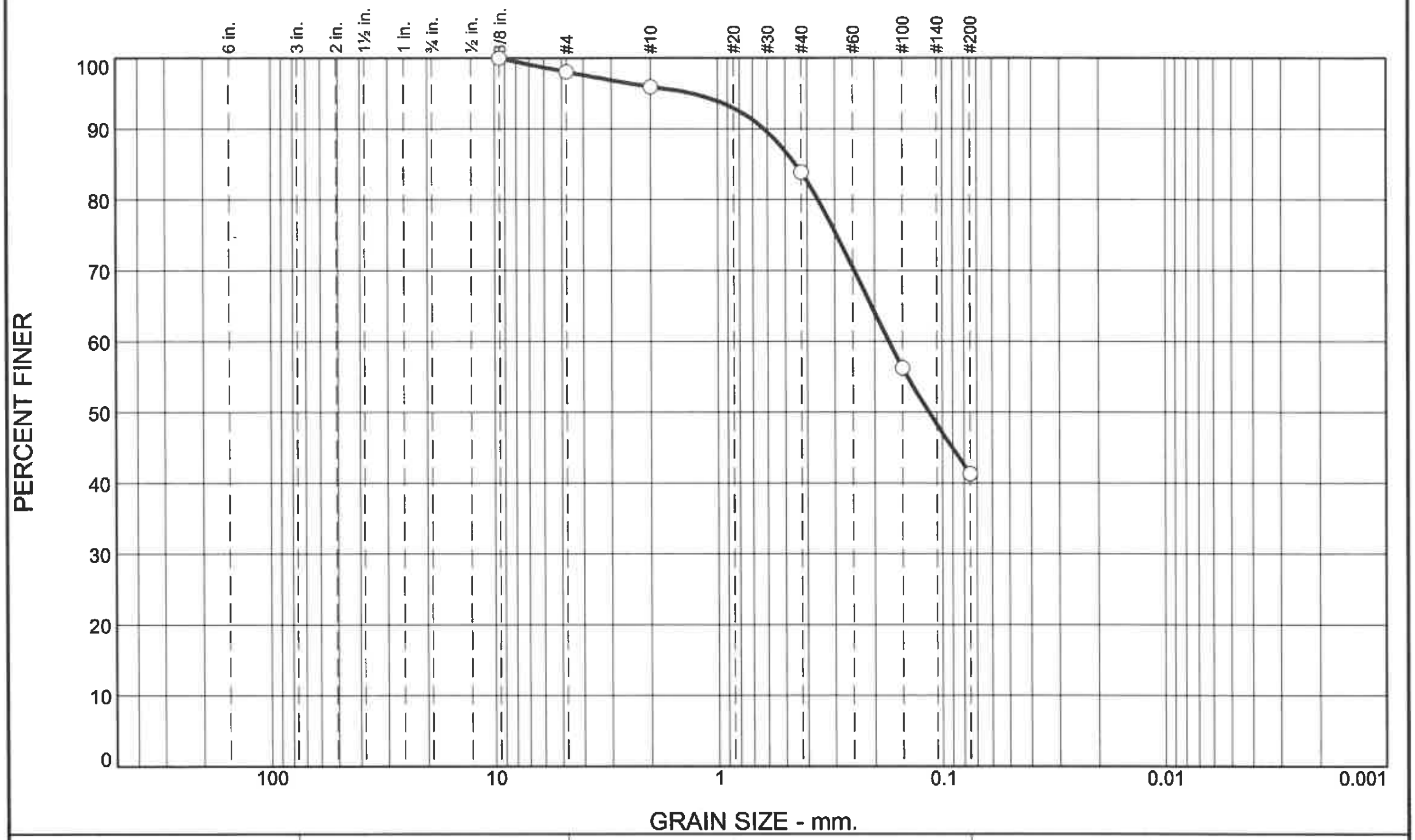
Plastic Limit Data

Run No.	1	2	3	4	
Wet+Tare	24.64				
Dry+Tare	22.62				
Tare	13.72				
Moisture	22.7				

Natural Moisture Data

Wet+Tare	Dry+Tare	Tare	Moisture
91.5	76.8	15.6	24.0

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	1.9	2.1	12.1	42.6	41.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8"	100.0		
#4	98.1		
#10	96.0		
#40	83.9		
#100	56.2		
#200	41.3		

* (no specification provided)

Material Description

Silty SAND

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.6209 D_{85}= 0.4497 D_{60}= 0.1730
D_{50}= 0.1149 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Source of Sample: AB-03 / S-2 **Depth:** 5
Sample Number: 8385-4
Date: 2/18/21

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Buttenweiser Residence
Project No: 08-175/200631 **Figure**

Tested By: AD **Checked By:** JAM

GRAIN SIZE DISTRIBUTION TEST DATA

2/18/2021

Client: Aspect Consulting
Project: Buttenweiser Residence
Project Number: 08-175/200631
Location: AB-03 / S-2
Depth: 5
Sample Number: 8385-4
Material Description: Silty SAND
Date: 2/18/21
USCS Classification: SM
Tested by: AD
Checked by: JAM

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 130.00
Tare Wt. = 12.70
Minus #200 from wash = 40.7%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
210.60	12.70	0.00	3/8"	0.00	100.0
			#4	3.80	98.1
			#10	8.00	96.0
			#40	31.90	83.9
			#100	86.60	56.2
			#200	116.10	41.3

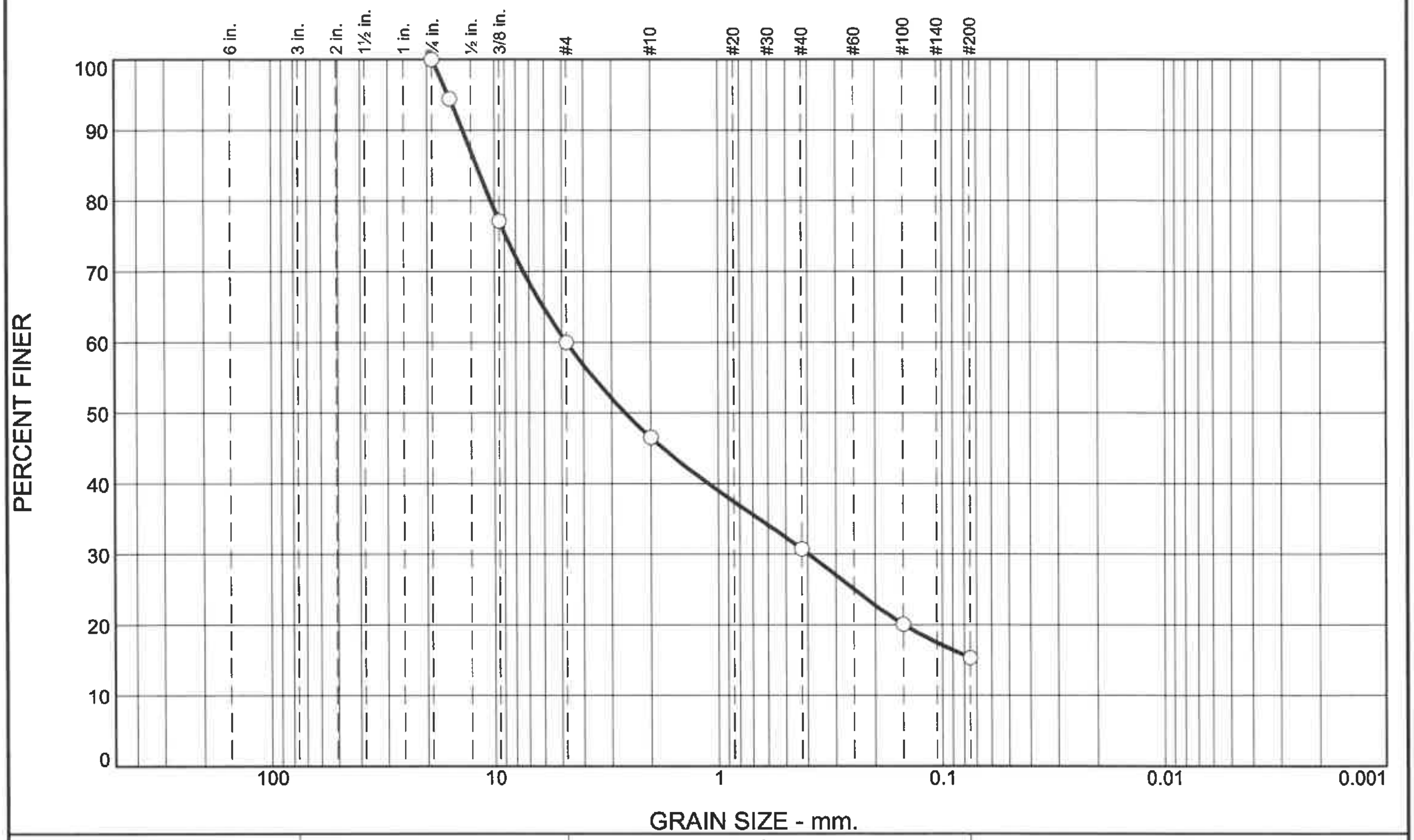
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	1.9	1.9	2.1	12.1	42.6	56.8			41.3

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
						0.1149	0.1730	0.3578	0.4497	0.6209	1.2988

Fineness Modulus
0.90

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	40.0	13.4	15.9	15.4	15.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
5/8"	94.5		
3/8"	77.2		
#4	60.0		
#10	46.6		
#40	30.7		
#100	20.1		
#200	15.3		

* (no specification provided)

Material Description

Silty SAND with gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 13.8818	D_{85}= 12.0204	D_{60}= 4.7548
D_{50}= 2.5984	D_{30}= 0.3963	D_{15}=
D_{10}=	C_u=	C_c=

Classification

USCS= SM AASHTO=

Remarks

Source of Sample: AB-03 / S-5 **Depth:** 15
Sample Number: 8385-6

Date: 2/18/21

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Buttenweiser Residence
Project No: 08-175/200631 **Figure**

Tested By: AD **Checked By:** JAM

GRAIN SIZE DISTRIBUTION TEST DATA

2/18/2021

Client: Aspect Consulting
Project: Buttenweiser Residence
Project Number: 08-175/200631
Location: AB-03 / S-5
Depth: 15
Sample Number: 8385-6
Material Description: Silty SAND with gravel
Date: 2/18/21
USCS Classification: SM
Tested by: AD
Checked by: JAM

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 362.40
Tare Wt. = 16.10
Minus #200 from wash = 14.7%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
421.90	16.10	0.00	3/4"	0.00	100.0
			5/8"	22.50	94.5
			3/8"	92.70	77.2
			#4	162.40	60.0
			#10	216.90	46.6
			#40	281.10	30.7
			#100	324.30	20.1
			#200	343.70	15.3

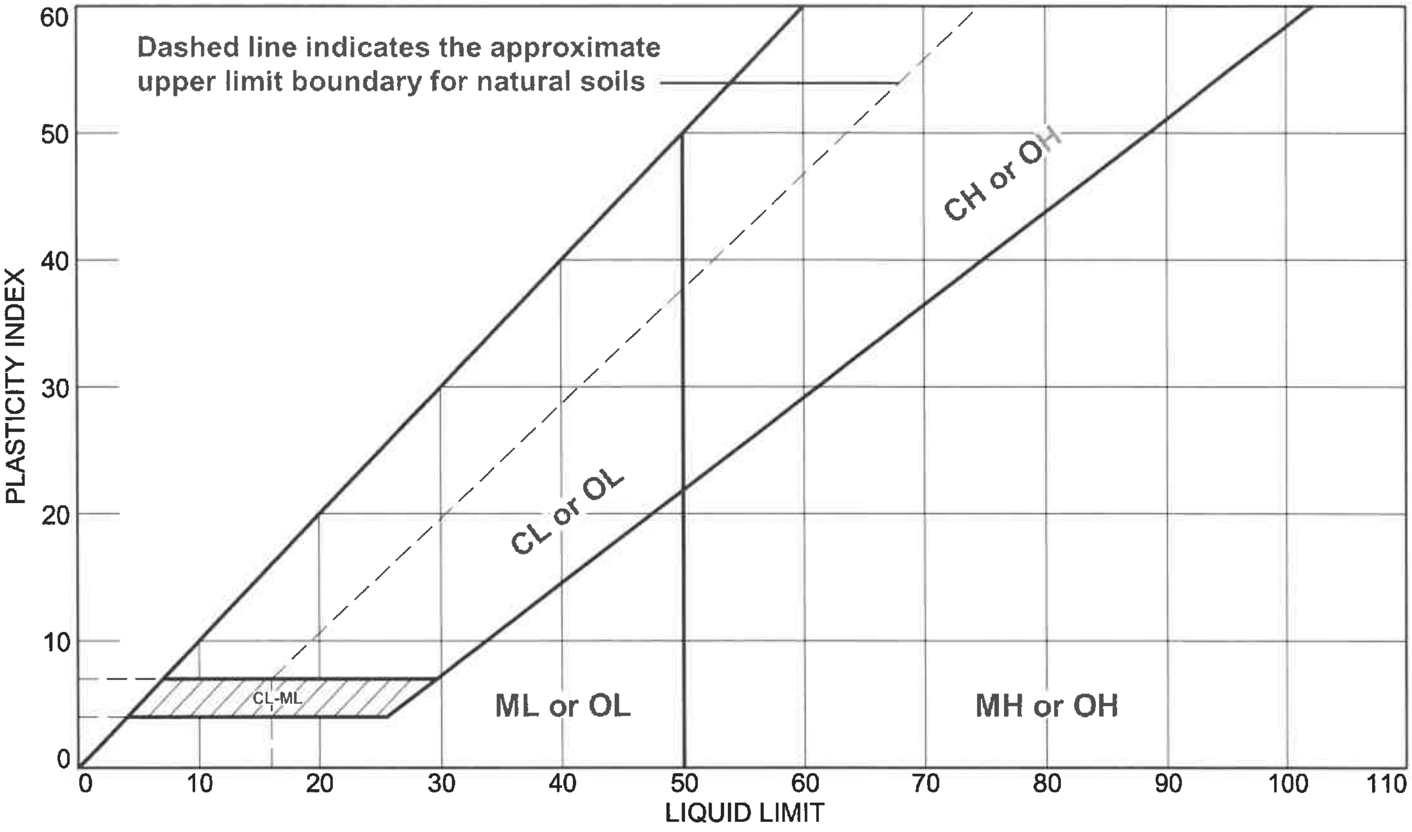
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	40.0	40.0	13.4	15.9	15.4	44.7			15.3

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
			0.1485	0.3963	1.1031	2.5984	4.7548	10.3921	12.0204	13.8818	16.1503

Fineness Modulus
3.92

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA

	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	LIQUIDITY INDEX	USCS
●	AB-04 / S-1	8385-8	2.5	31.3	NP	NV	NP		

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Buttenweiser Residence

Project No.: 08-175/200631 **Figure**

Tested By: AD **Checked By:** JAM

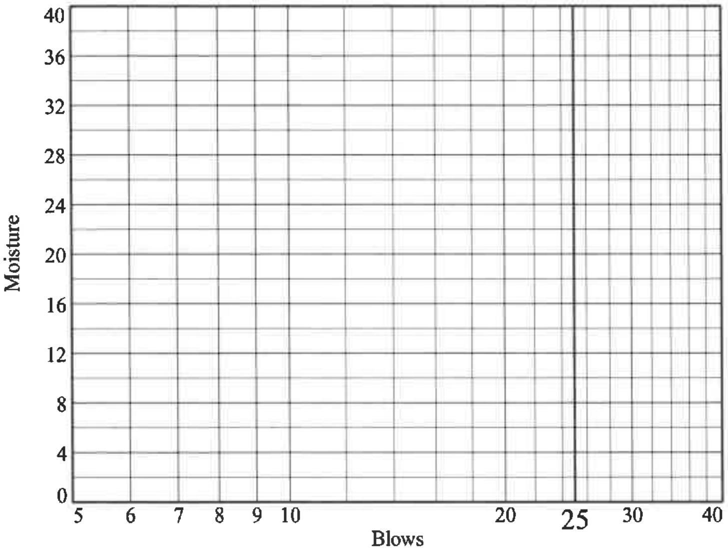
LIQUID AND PLASTIC LIMIT TEST DATA

2/18/2021

Client: Aspect Consulting
Project: Buttenweiser Residence
Project Number: 08-175/200631
Location: AB-04 / S-1
Depth: 2.5
Sample Number: 8385-8
Tested by: AD
Checked by: JAM

Liquid Limit Data

Run No.	1	2	3	4	5	6
Wet+Tare						
Dry+Tare						
Tare						
# Blows						
Moisture						



Liquid Limit= NV
Plastic Limit= NP
Plasticity Index= NP
Natural Moisture= 31.3

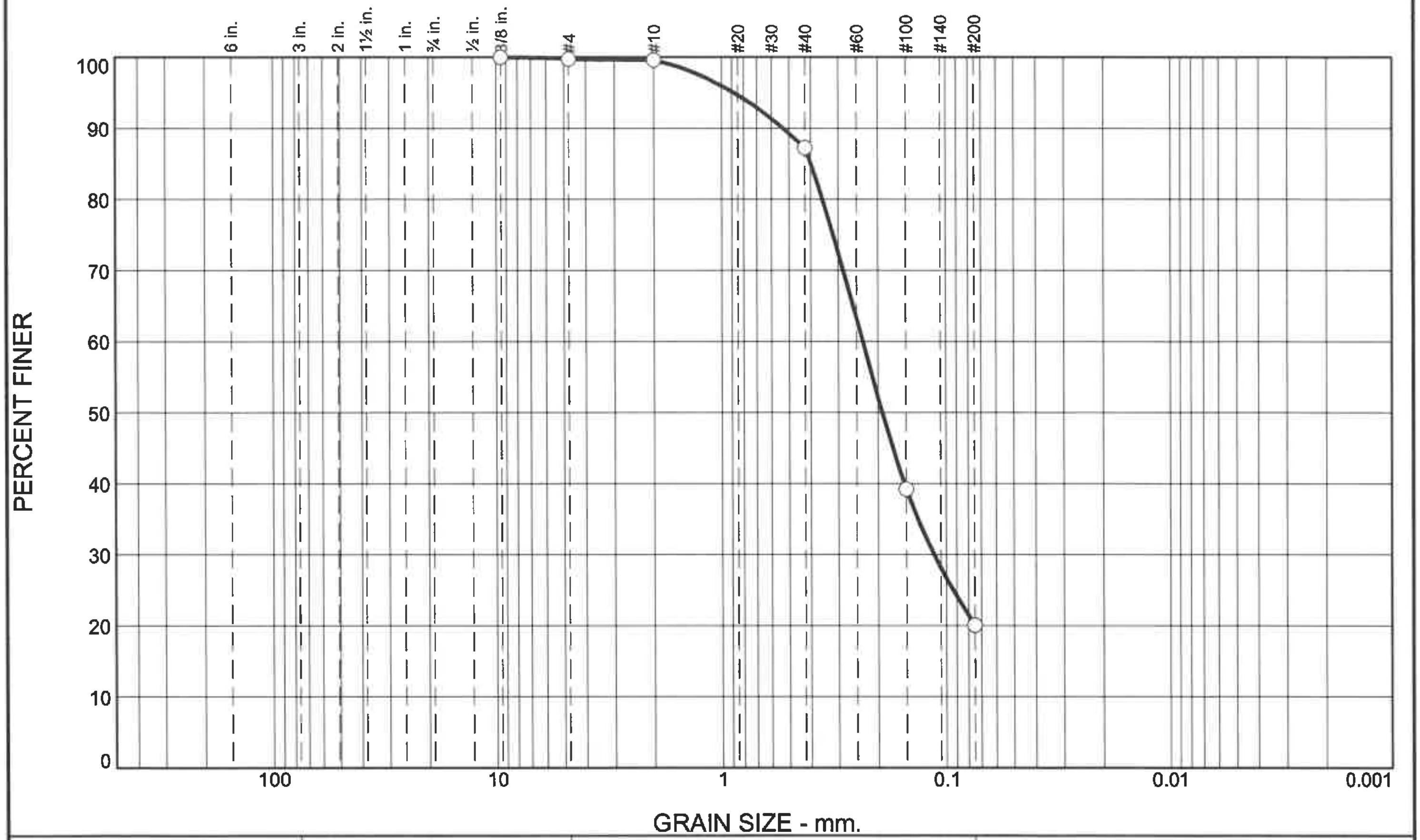
Plastic Limit Data

Run No.	1	2	3	4	
Wet+Tare					
Dry+Tare					
Tare					
Moisture					

Natural Moisture Data

Wet+Tare	Dry+Tare	Tare	Moisture
35.9	30.4	12.8	31.3

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.2	0.2	12.3	67.2	20.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8"	100.0		
#4	99.8		
#10	99.6		
#40	87.3		
#100	39.2		
#200	20.1		

* (no specification provided)

Material Description

Silty SAND

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.5316 D_{85}= 0.3996 D_{60}= 0.2344
D_{50}= 0.1917 D_{30}= 0.1135 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Source of Sample: AB-05 / S-2 **Depth:** 5
Sample Number: 8385-11

Date: 2/18/21

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Buttenweiser Residence

Project No: 08-175/200631 **Figure**

Tested By: AD **Checked By:** JAM

GRAIN SIZE DISTRIBUTION TEST DATA

2/18/2021

Client: Aspect Consulting
Project: Buttenweiser Residence
Project Number: 08-175/200631
Location: AB-05 / S-2
Depth: 5
Sample Number: 8385-11
Material Description: Silty SAND
Date: 2/18/21
USCS Classification: SM
Tested by: AD
Checked by: JAM

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 353.20
Tare Wt. = 12.70
Minus #200 from wash = 18.1%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
428.50	12.70	0.00	3/8"	0.00	100.0
			#4	1.00	99.8
			#10	1.60	99.6
			#40	53.00	87.3
			#100	252.70	39.2
			#200	332.40	20.1

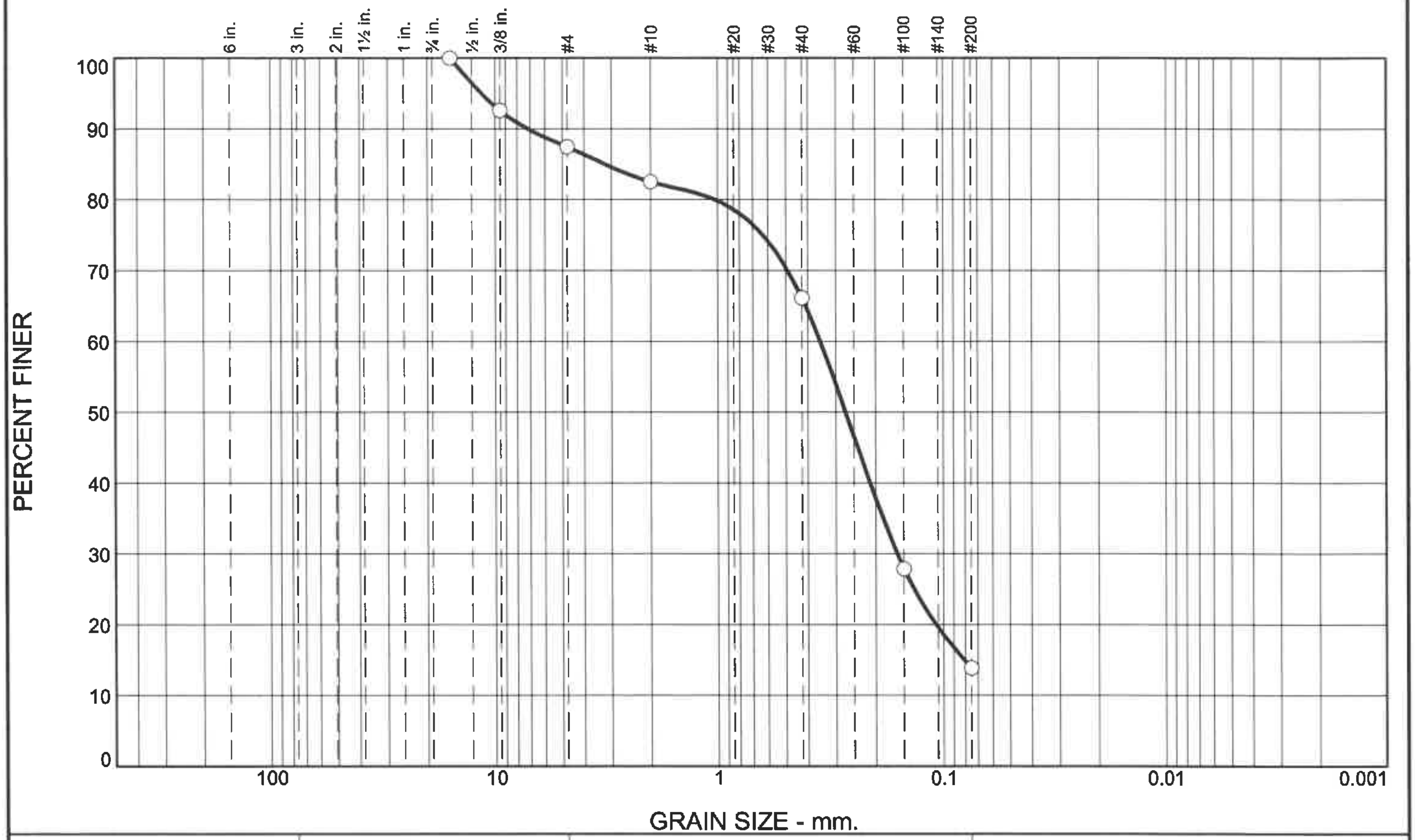
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	0.2	0.2	0.2	12.3	67.2	79.7			20.1

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
				0.1135	0.1530	0.1917	0.2344	0.3538	0.3996	0.5316	0.8803

Fineness Modulus
1.01

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	12.5	4.9	16.5	52.2	13.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
5/8"	100.0		
3/8"	92.6		
#4	87.5		
#10	82.6		
#40	66.1		
#100	27.9		
#200	13.9		

* (no specification provided)

Material Description

Silty SAND

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 7.1375 D_{85}= 3.1836 D_{60}= 0.3530
D_{50}= 0.2721 D_{30}= 0.1607 D_{15}= 0.0807
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Source of Sample: AB-06 / S-2 **Depth:** 5
Sample Number: 8385-14

Date: 2/18/21

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Buttenweiser Residence
Project No: 08-175/200631 **Figure**

Tested By: AD **Checked By:** JAM

GRAIN SIZE DISTRIBUTION TEST DATA

2/18/2021

Client: Aspect Consulting
Project: Buttenweiser Residence
Project Number: 08-175/200631
Location: AB-06 / S-2
Depth: 5
Sample Number: 8385-14
Material Description: Silty SAND
Date: 2/18/21
USCS Classification: SM
Tested by: AD
Checked by: JAM

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 337.90
Tare Wt. = 12.70
Minus #200 from wash = 12.4%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
384.10	12.70	0.00	5/8"	0.00	100.0
			3/8"	27.40	92.6
			#4	46.50	87.5
			#10	64.80	82.6
			#40	125.80	66.1
			#100	267.90	27.9
			#200	319.90	13.9

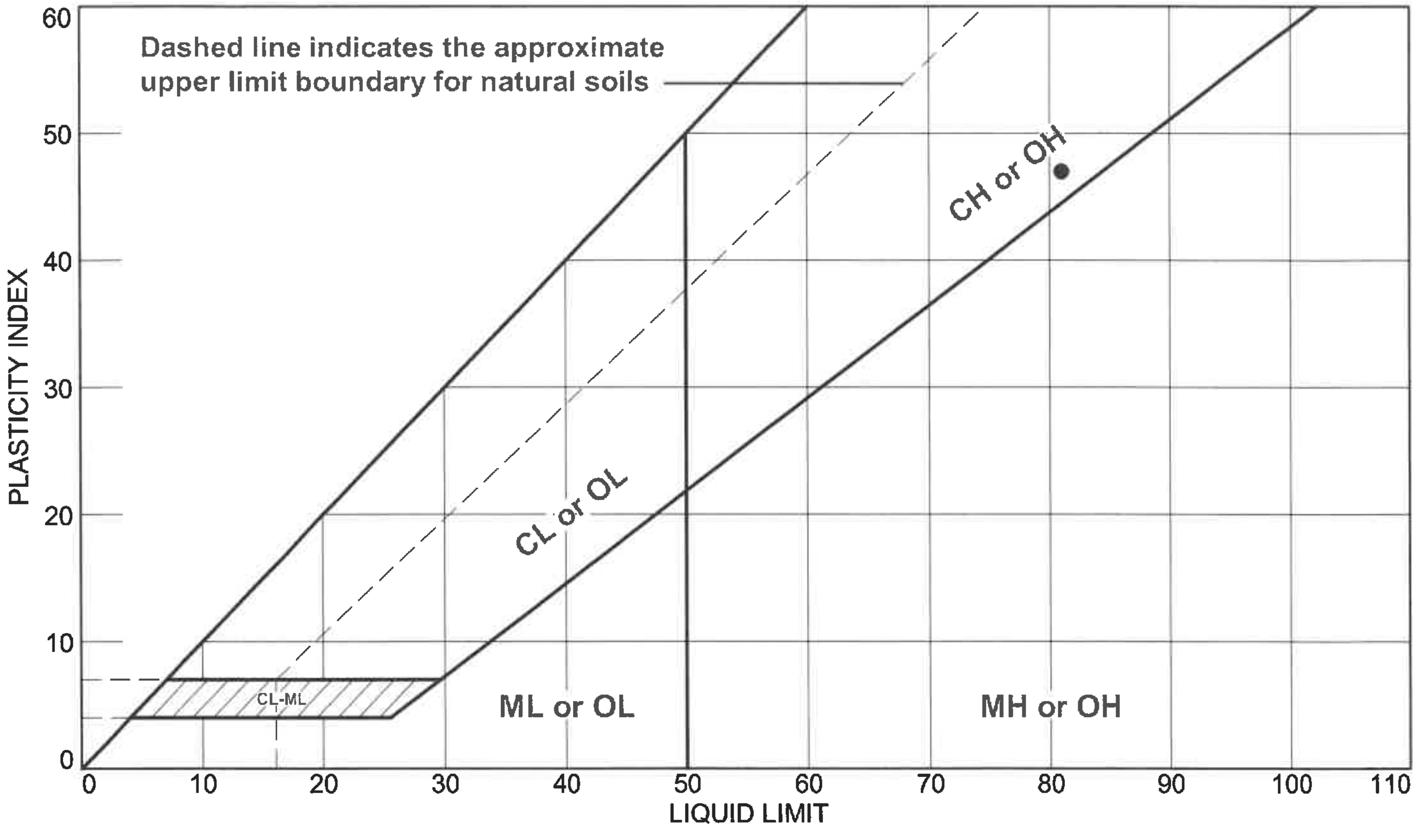
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	12.5	12.5	4.9	16.5	52.2	73.6			13.9

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
		0.0807	0.1081	0.1607	0.2119	0.2721	0.3530	1.0200	3.1836	7.1375	11.4671

Fineness Modulus
2.00

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA

	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	LIQUIDITY INDEX	USCS
●	AB-06 / S-5	8385-16	15	37.4	34	81	47	0.1	CH

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Buttenweiser Residence

Project No.: 08-175/200631

Figure

Tested By: AD

Checked By: JAM

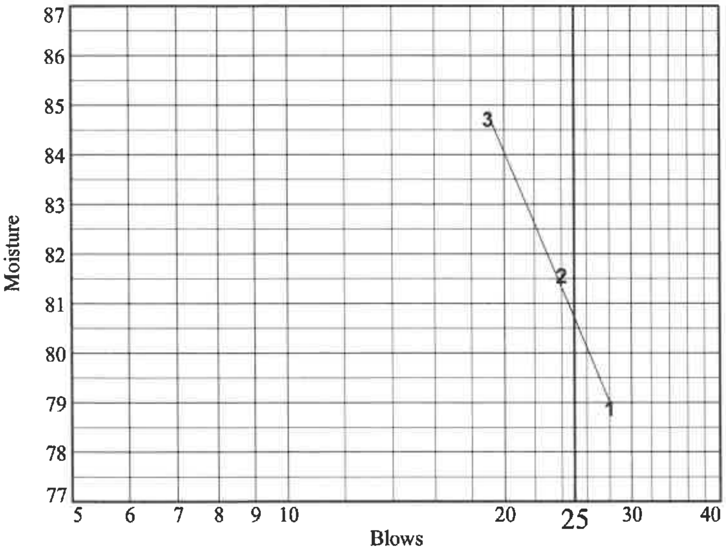
LIQUID AND PLASTIC LIMIT TEST DATA

2/18/2021

Client: Aspect Consulting
Project: Buttenweiser Residence
Project Number: 08-175/200631
Location: AB-06 / S-5
Depth: 15
Sample Number: 8385-16
Material Description: Fat CLAY
USCS: CH
Tested by: AD
Checked by: JAM

Liquid Limit Data

Run No.	1	2	3	4	5	6
Wet+Tare	27.21	26.29	26.61			
Dry+Tare	21.27	20.67	20.73			
Tare	13.74	13.78	13.79			
# Blows	28	24	19			
Moisture	78.9	81.6	84.7			



Liquid Limit= 81
Plastic Limit= 34
Plasticity Index= 47
Natural Moisture= 37.4
Liquidity Index= 0.1

Plastic Limit Data

Run No.	1	2	3	4	
Wet+Tare	22.23				
Dry+Tare	20.05				
Tare	13.71				
Moisture	34.4				

Natural Moisture Data

Wet+Tare	Dry+Tare	Tare	Moisture
94.3	72.1	12.7	37.4

hayremcelroy
& associates, llc

9644 153rd Ave NE
Redmond, WA 98052
425.869.6750

Moisture Content

ASTM D-2216

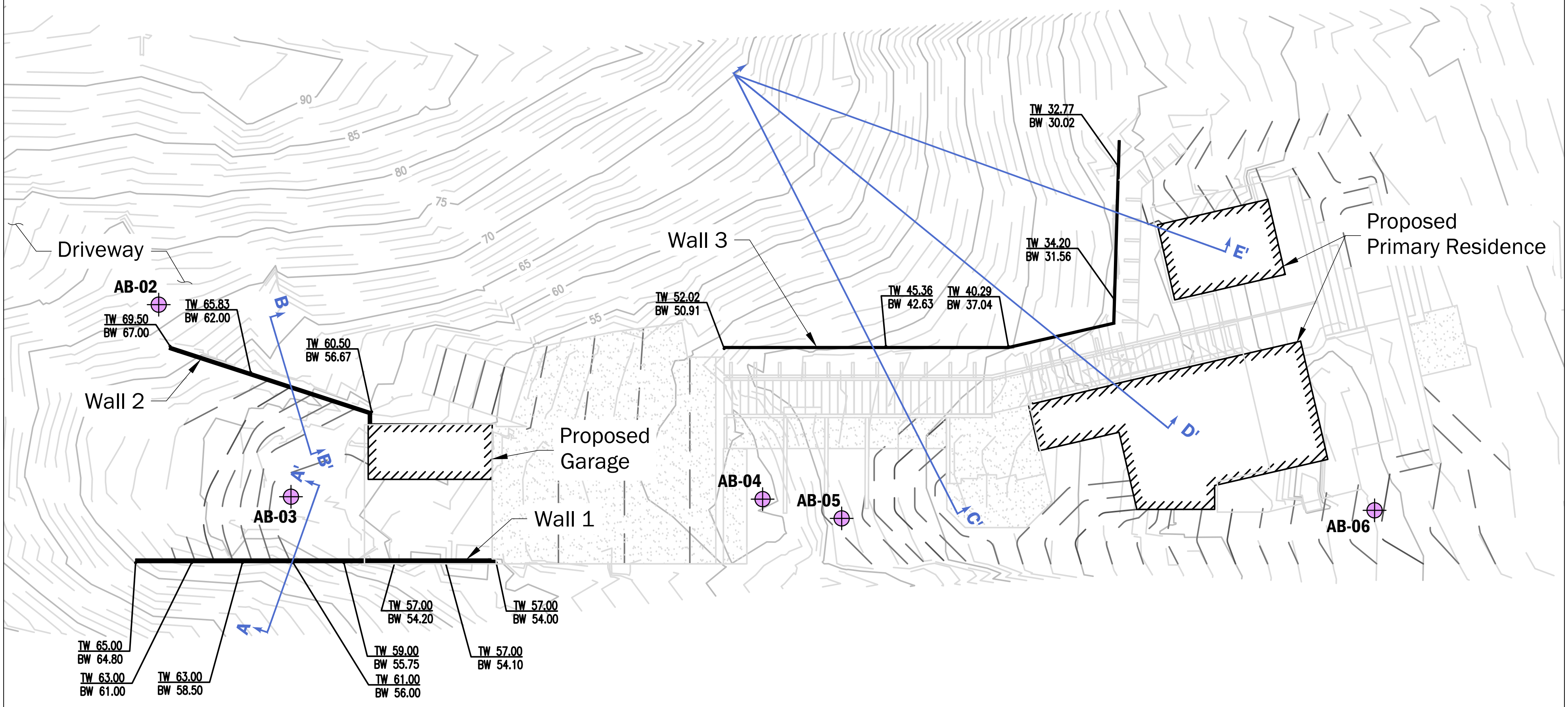
Project Number: 08-175/200631
Project Name: Buttenweiser Residence
Lab Number: 8385

Received Date: 2/5/2021
Start Date: 2/5/2021
Finish Date: 2/18/2021
Technician: AD

HMA Lab #	Boring	Sample	Depth (ft)	Weight of Moist Soil + Tare (g)	Weight of Dry Soil + Tare (g)	Tare Weight (g)	**Moisture Content (%)**
8385-1	AB-02	S-1	2.5	182.5	155.4	12.5	**19.0**
8385-2	AB-02	S-2	5	263.2	221.4	15.8	**20.3**
8385-3	AB-02	S-3	7.5	91.5	76.8	15.6	**24.0**
8385-4	AB-03	S-2	5	269.6	210.6	12.7	**29.8**
8385-5	AB-03	S-3	7.5	194.4	162.6	12.7	**21.2**
8385-6	AB-03	S-5	15	459.2	421.9	16.1	**9.2**
8385-7	AB-03	S-6	20	274.2	220.6	12.5	**25.8**
8385-8	AB-04	S-1	2.5	35.9	30.4	12.8	**31.3**
8385-9	AB-04	S-3	7.5	431.7	358.3	15.9	**21.4**
8385-10	AB-04	S-6	20	242.1	205.2	12.7	**19.2**
8385-11	AB-05	S-2	5	520.7	428.5	12.7	**22.2**
8385-12	AB-05	S-5	15	158.6	126.8	12.6	**27.8**
8385-13	AB-05	S-6	20	185.9	144.0	12.7	**31.9**
8385-14	AB-06	S-2	5	451.4	384.1	12.7	**18.1**
8385-15	AB-06	S-4	10	187.0	139.8	16.0	**38.1**
8385-16	AB-06	S-5	15	94.3	72.1	12.7	**37.4**

APPENDIX C

Wall Global Stability Analyses



Legend

⊕ **AB-XX** Boring Location

Existing 1' & 5' Topographic Contours

Proposed 1' & 5' Topographic Contours

Proposed Retaining Wall

A A' Cross Section for Global Stability Analysis

0 30 60 Feet

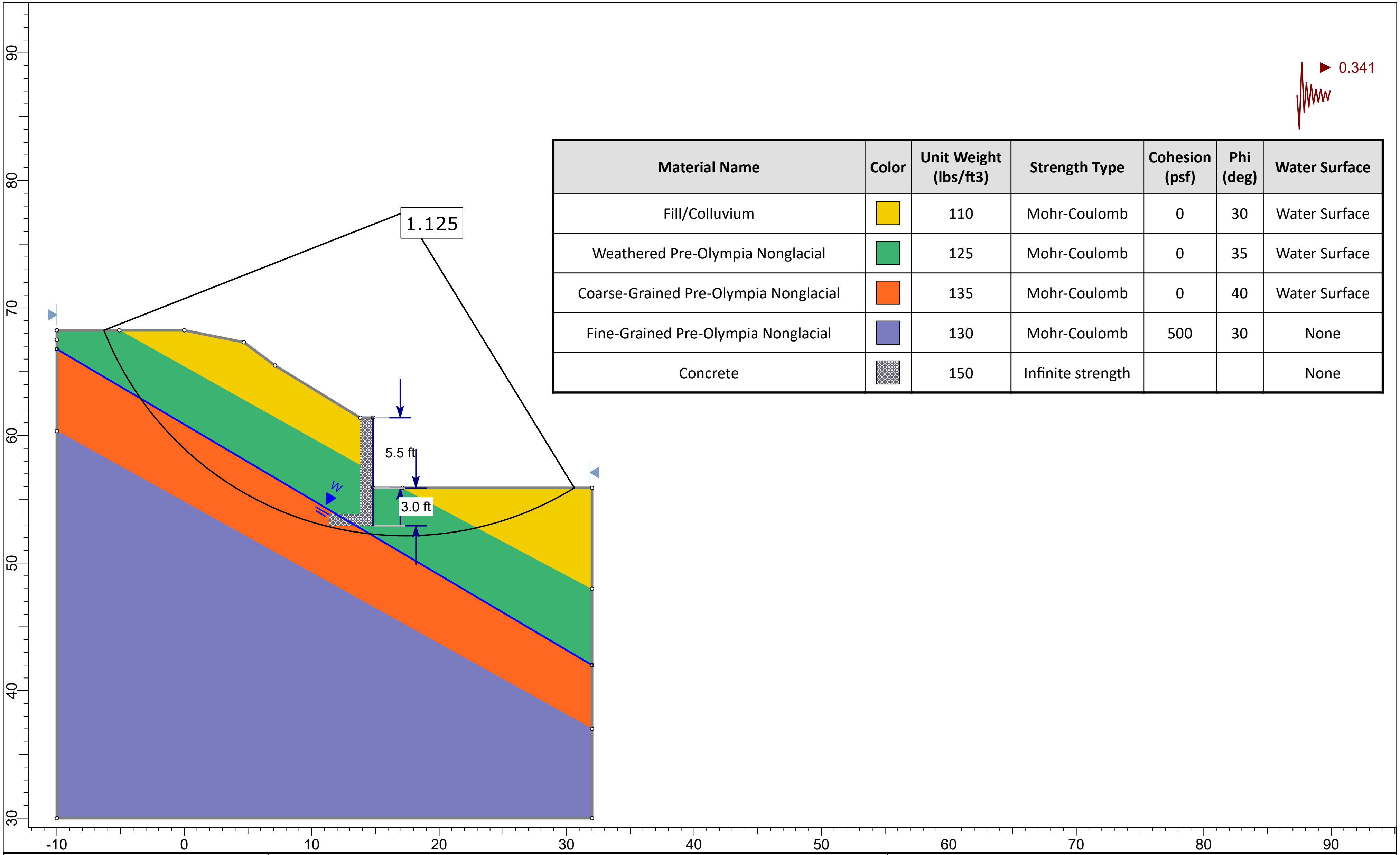
Global Stability Analysis Plan

Geotechnical Engineering Report
Buttenwieser/Wiley Residence
6838 96th Avenue SE
Mercer Island, Washington

Aspect CONSULTING

----	BY: MBR	FIGURE NO. **C-1**
PROJECT NO. 200631	REVISED BY: -	

CAD Path: Q:_GeoTech\200631 Buttenwieser\ButtenwieserGlobalStability.dwg 8.5x11 Landscape || Date Saved: Sep 01, 2021 2:09pm || User: mreiter



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Fill/Colluvium	🟨	110	Mohr-Coulomb	0	30	Water Surface
Weathered Pre-Olympia Nonglacial	🟩	125	Mohr-Coulomb	0	35	Water Surface
Coarse-Grained Pre-Olympia Nonglacial	🟧	135	Mohr-Coulomb	0	40	Water Surface
Fine-Grained Pre-Olympia Nonglacial	🟪	130	Mohr-Coulomb	500	30	None
Concrete	▨	150	Infinite strength			None

Legend

⬚	Search Grid
⧓	Search Limits
▼	Modeled Groundwater Level
B-2	Boring Location and Depth

Section A-A'
Seismic

Global Stability Analysis

Geotechnical Engineering Report
Buttenwieser/Wiley Residence
Mercer Island, WA

SLIDEINTERPRET 8.032

SCALE: 1:125

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Aspect CONSULTING

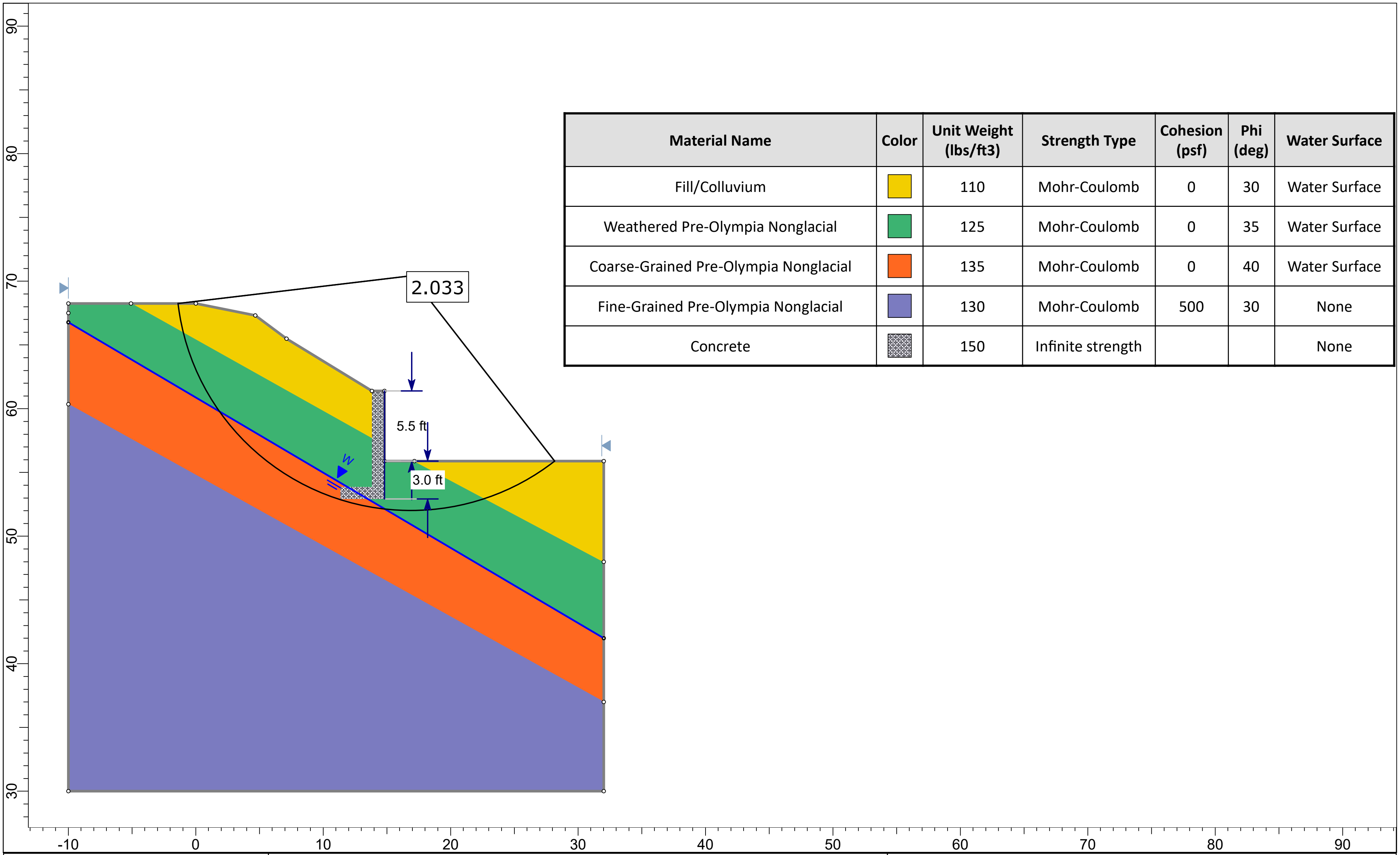
9/1/2021

PROJECT NO.
200631

BY:
MBR

REVIEWED BY:
ETB/HHH

APPENDIX:
C-2



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Fill/Colluvium	🟨	110	Mohr-Coulomb	0	30	Water Surface
Weathered Pre-Olympia Nonglacial	🟩	125	Mohr-Coulomb	0	35	Water Surface
Coarse-Grained Pre-Olympia Nonglacial	🟧	135	Mohr-Coulomb	0	40	Water Surface
Fine-Grained Pre-Olympia Nonglacial	🟪	130	Mohr-Coulomb	500	30	None
Concrete	▨	150	Infinite strength			None

Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- B-2 Boring Location and Depth

Section A-A'
Static

Global Stability Analysis

Geotechnical Engineering Report
Buttenwieser/Wiley Residence
Mercer Island, WA

SLIDEINTERPRET 8.032

SCALE: 1:125

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Aspect CONSULTING

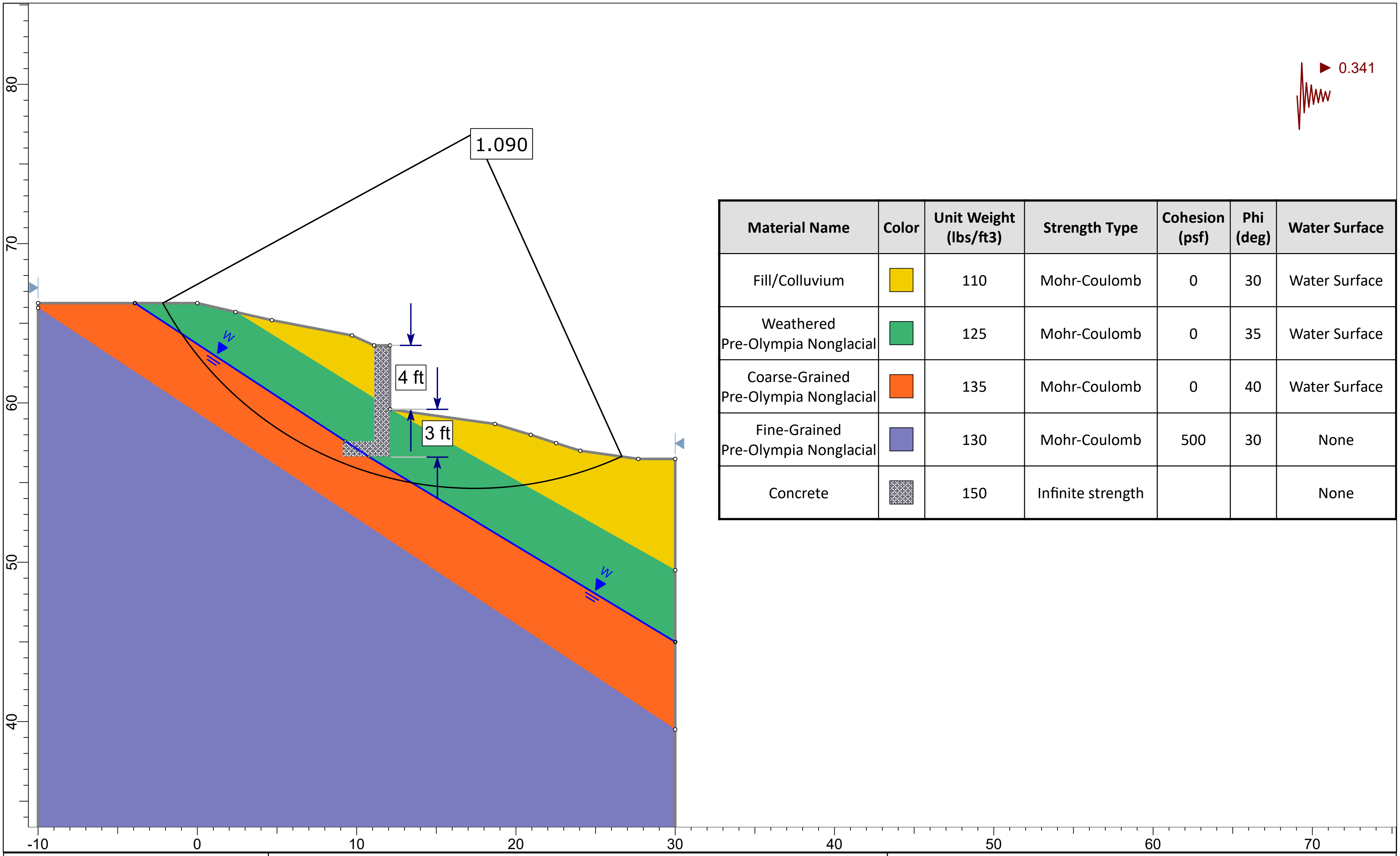
9/1/2021

PROJECT NO.
200631

BY:
MBR

REVIEWED BY:
ETB/HHH

APPENDIX:
C-3



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Fill/Colluvium	🟨	110	Mohr-Coulomb	0	30	Water Surface
Weathered Pre-Olympia Nonglacial	🟩	125	Mohr-Coulomb	0	35	Water Surface
Coarse-Grained Pre-Olympia Nonglacial	🟧	135	Mohr-Coulomb	0	40	Water Surface
Fine-Grained Pre-Olympia Nonglacial	🟪	130	Mohr-Coulomb	500	30	None
Concrete	▨	150	Infinite strength			None

Legend

Search Grid

Search Limits

Modeled Groundwater Level

Boring Location and Depth

Section B-B'
Seismic

Global Stability Analysis

Geotechnical Engineering Report
Buttenwieser/Wiley Residence
Mercer Island, WA

SLIDEINTERPRET 8.032

SCALE: 1:100

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9/1/2021

PROJECT NO.
200631

BY:
MBR

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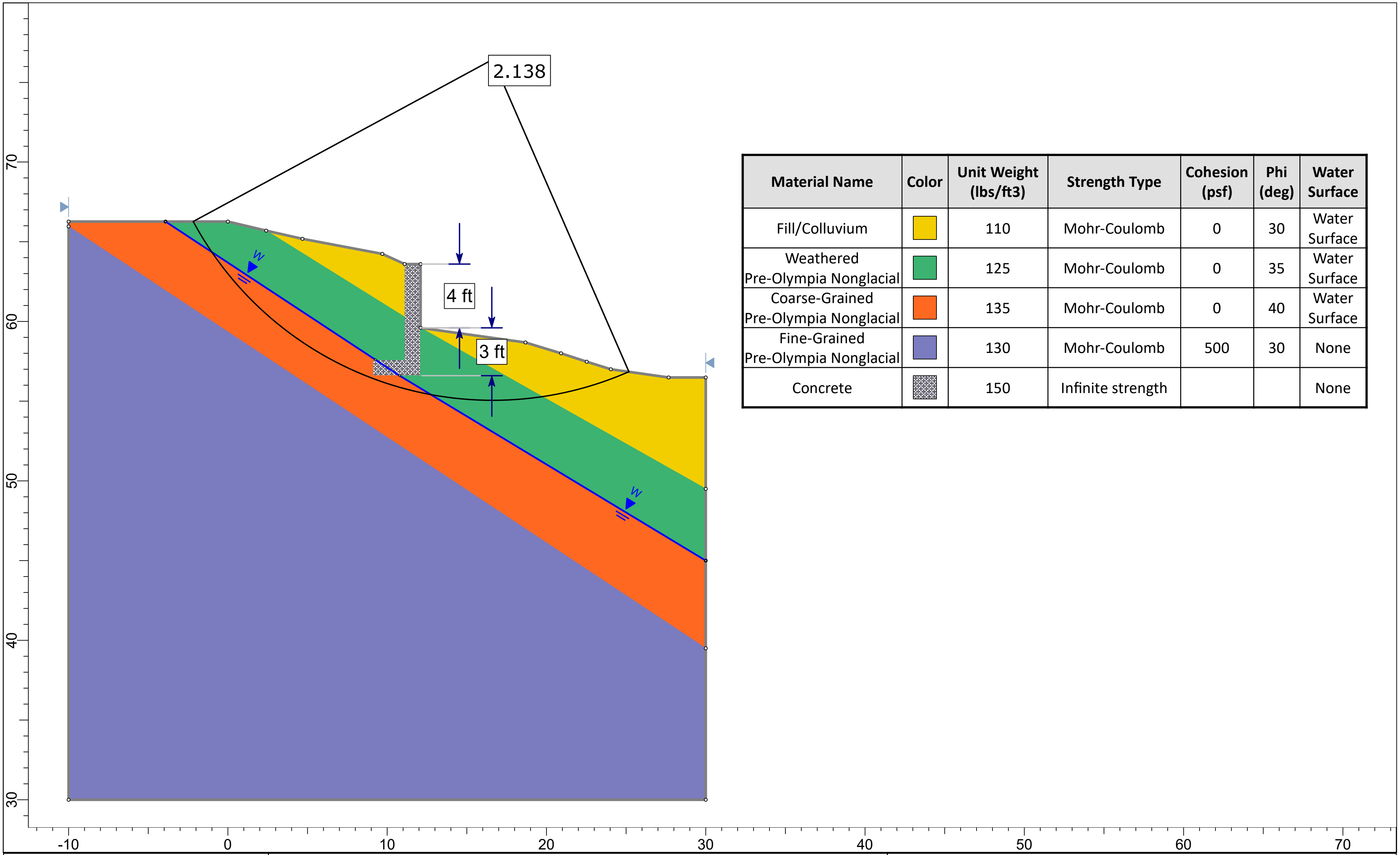
ETB/HHH

APPENDIX:
C-4



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Fill/Colluvium	🟨	110	Mohr-Coulomb	0	30	Water Surface
Weathered Pre-Olympia Nonglacial	🟩	125	Mohr-Coulomb	0	35	Water Surface
Coarse-Grained Pre-Olympia Nonglacial	🟧	135	Mohr-Coulomb	0	40	Water Surface
Fine-Grained Pre-Olympia Nonglacial	🟪	130	Mohr-Coulomb	500	30	None
Concrete	▨	150	Infinite strength			None

Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

Section B-B'
Static

Global Stability Analysis

Geotechnical Engineering Report
Buttenwieser/Wiley Residence
Mercer Island, WA

SLIDEINTERPRET 8.032

SCALE: 1:100

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9/1/2021

PROJECT NO.
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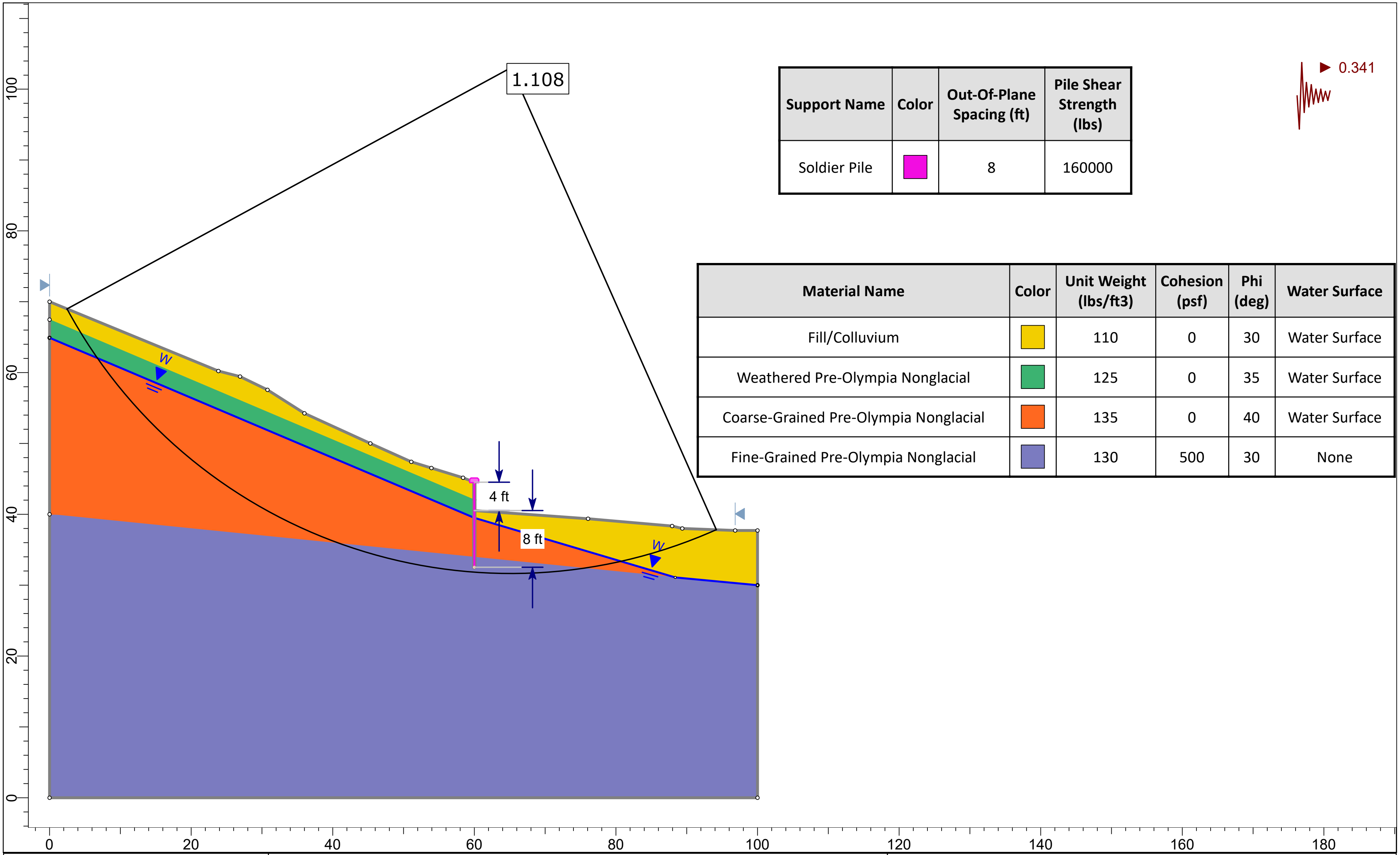
REVIEWED BY:
ETB/HHH

APPENDIX:
C-5



Support Name	Color	Out-Of-Plane Spacing (ft)	Pile Shear Strength (lbs)
Soldier Pile	🟪	8	160000

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)	Water Surface
Fill/Colluvium	🟨	110	0	30	Water Surface
Weathered Pre-Olympia Nonglacial	🟩	125	0	35	Water Surface
Coarse-Grained Pre-Olympia Nonglacial	🟧	135	0	40	Water Surface
Fine-Grained Pre-Olympia Nonglacial	🟪	130	500	30	None

Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

Section C-C'
Seismic

Global Stability Analysis

Geotechnical Engineering Report
Buttenwieser/Wiley Residence
Mercer Island, WA

SLIDEINTERPRET 8.032

SCALE: 1:225

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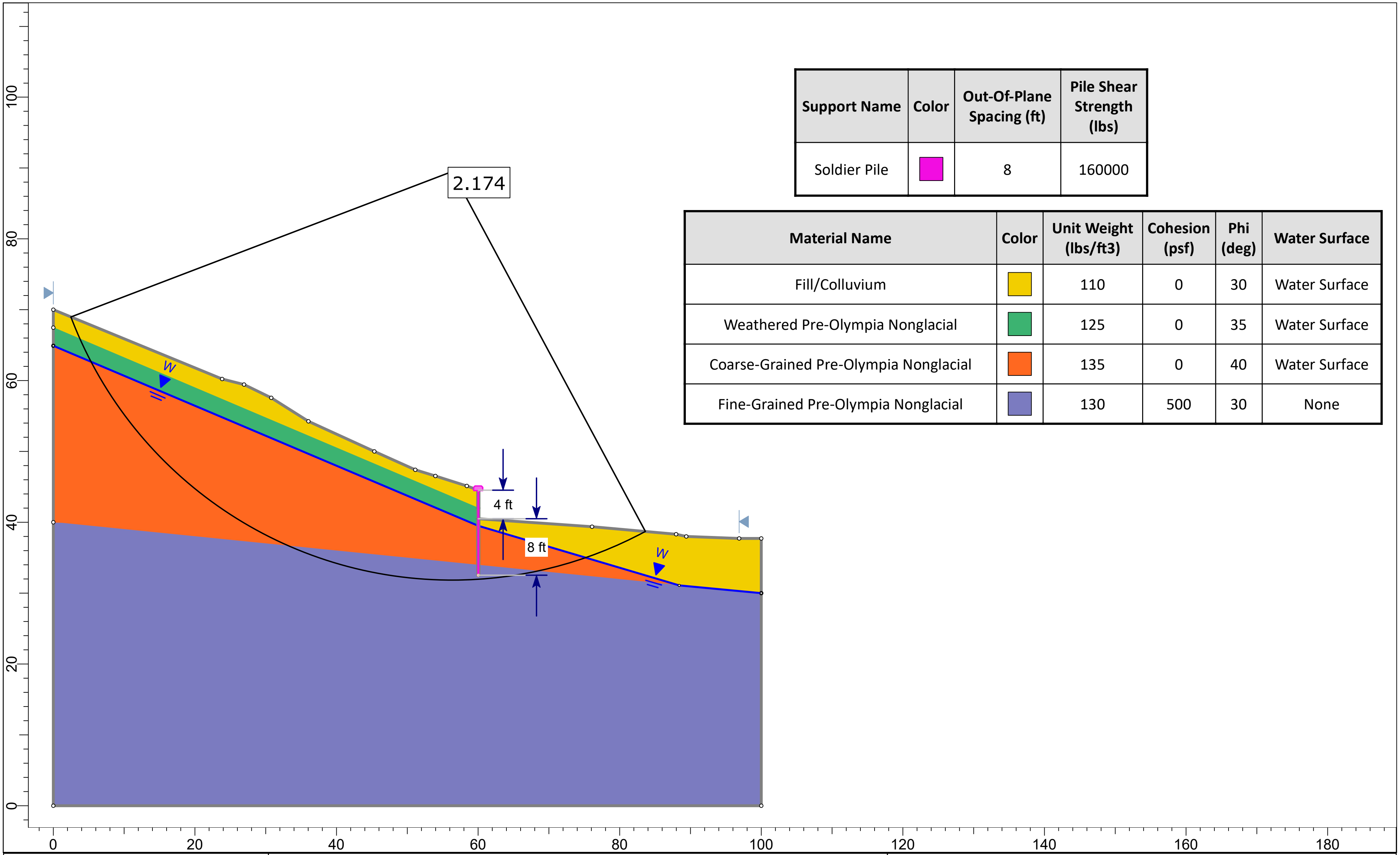
9/1/2021	BY: MBR	APPENDIX:
PROJECT NO. 200631	REVIEWED BY: ETB/HHH	C-6



Support Name	Color	Out-Of-Plane Spacing (ft)	Pile Shear Strength (lbs)
Soldier Pile	🟪	8	160000

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)	Water Surface
Fill/Colluvium	🟨	110	0	30	Water Surface
Weathered Pre-Olympia Nonglacial	🟩	125	0	35	Water Surface
Coarse-Grained Pre-Olympia Nonglacial	🟧	135	0	40	Water Surface
Fine-Grained Pre-Olympia Nonglacial	🟦	130	500	30	None

Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

Section C-C'
Static

Global Stability Analysis

Geotechnical Engineering Report
Buttenwieser/Wiley Residence
Mercer Island, WA

SLIDEINTERPRET 8.032

SCALE: 1:225

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200631

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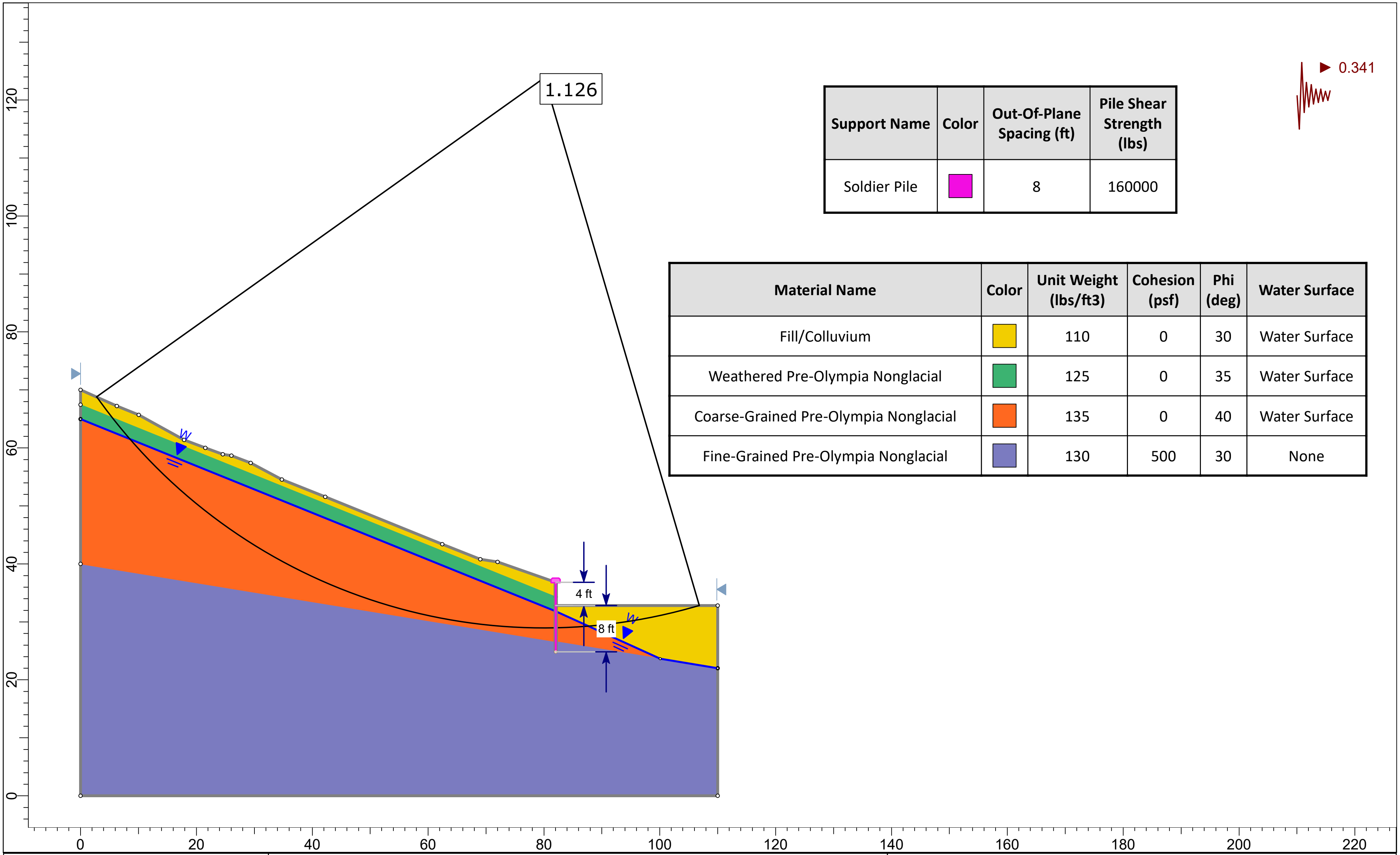
APPENDIX:
C-7



Support Name	Color	Out-Of-Plane Spacing (ft)	Pile Shear Strength (lbs)
Soldier Pile	🟪	8	160000

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)	Water Surface
Fill/Colluvium	🟨	110	0	30	Water Surface
Weathered Pre-Olympia Nonglacial	🟩	125	0	35	Water Surface
Coarse-Grained Pre-Olympia Nonglacial	🟧	135	0	40	Water Surface
Fine-Grained Pre-Olympia Nonglacial	🟦	130	500	30	None

Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

Section D-D'
Seismic

Global Stability Analysis

Geotechnical Engineering Report
Buttenwieser/Wiley Residence
Mercer Island, WA

SLIDEINTERPRET 8.032

SCALE: 1:275

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PROJECT NO.
200631

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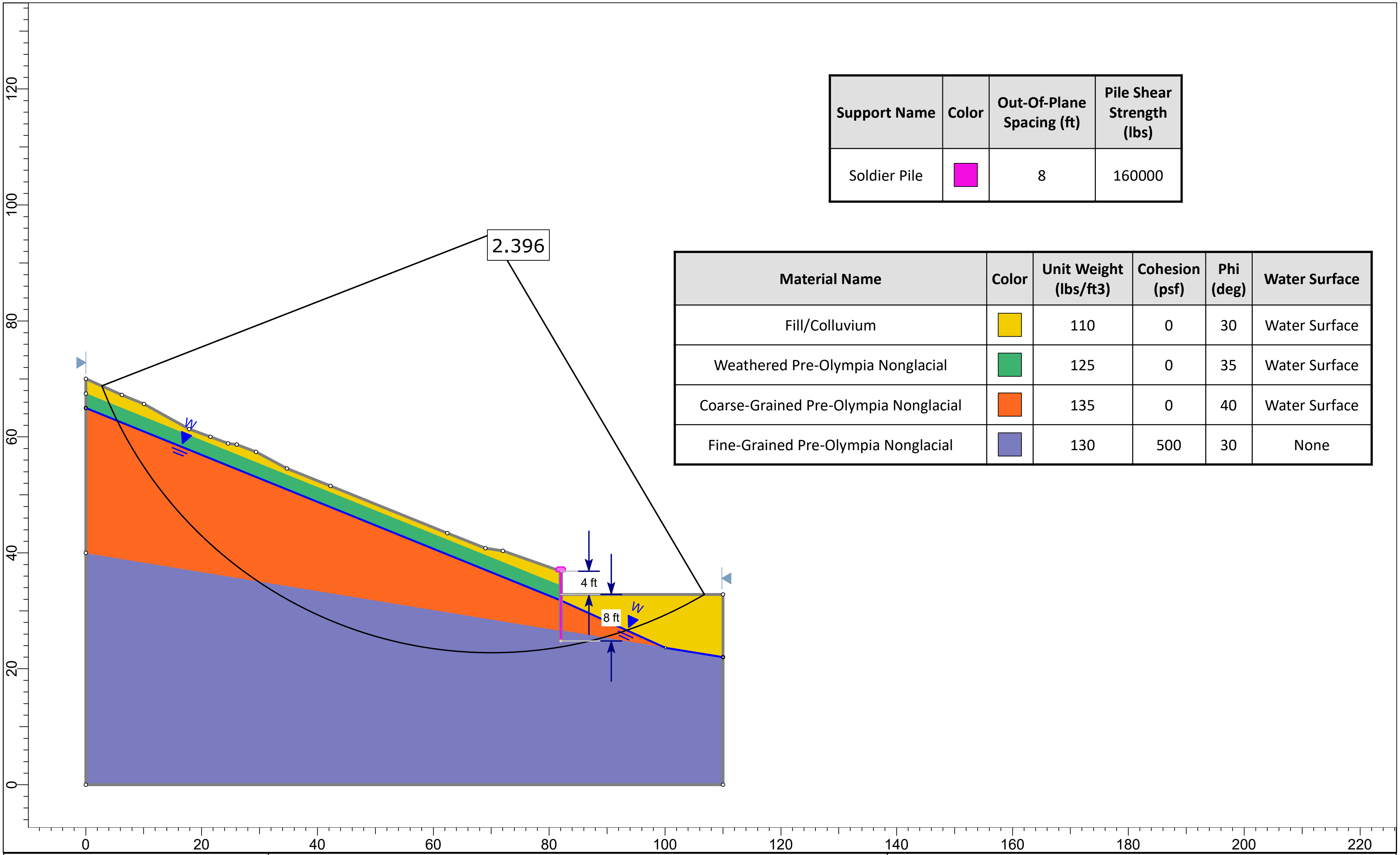
APPENDIX:
C-8



Support Name	Color	Out-Of-Plane Spacing (ft)	Pile Shear Strength (lbs)
Soldier Pile	🟪	8	160000

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)	Water Surface
Fill/Colluvium	🟨	110	0	30	Water Surface
Weathered Pre-Olympia Nonglacial	🟩	125	0	35	Water Surface
Coarse-Grained Pre-Olympia Nonglacial	🟧	135	0	40	Water Surface
Fine-Grained Pre-Olympia Nonglacial	🟦	130	500	30	None

Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

Section D-D'
Static

Global Stability Analysis

Geotechnical Engineering Report
Buttenwieser/Wiley Residence
Mercer Island, WA

SLIDEINTERPRET 8.032

SCALE: 1:275

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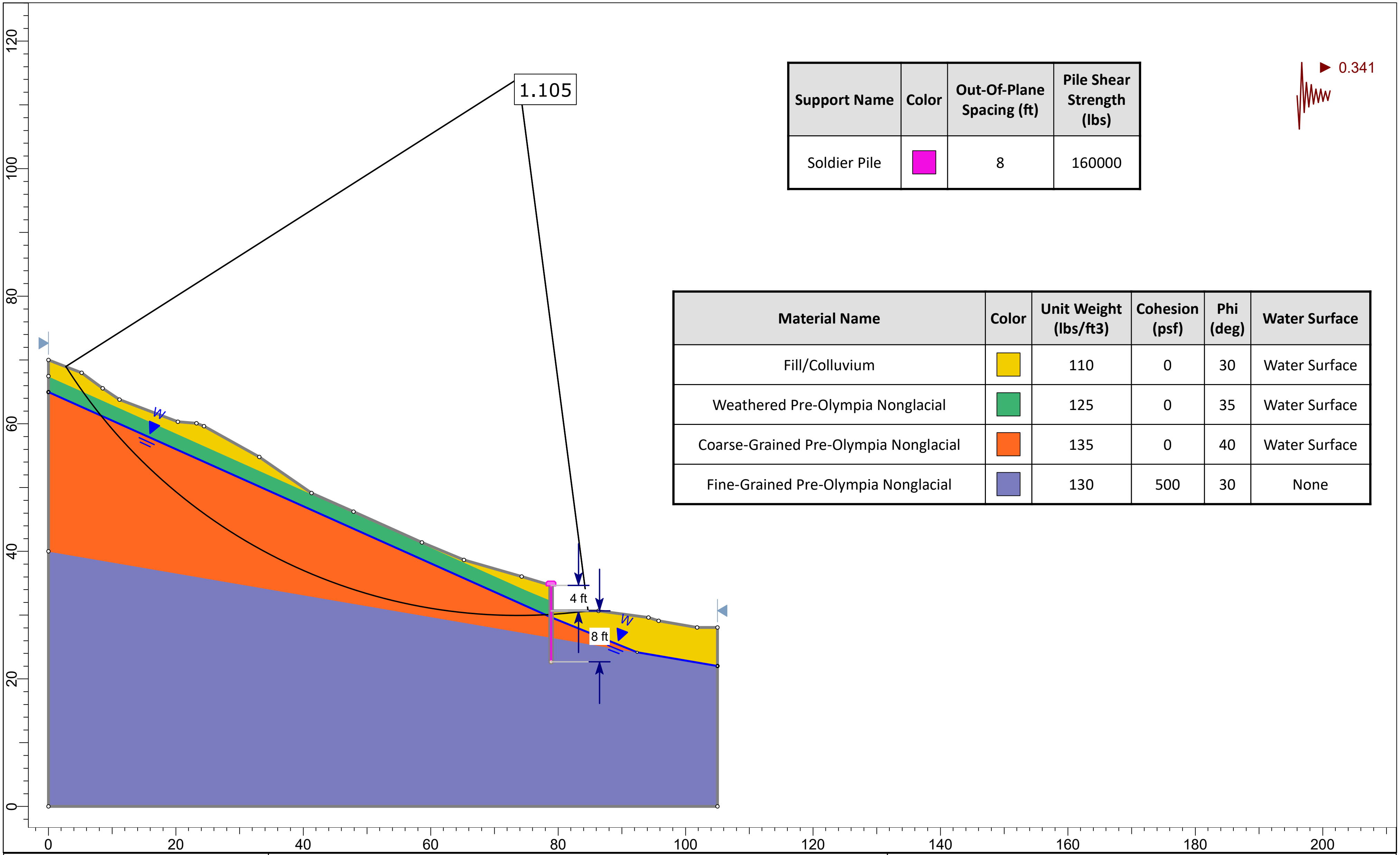
APPENDIX:
C-9



Support Name	Color	Out-Of-Plane Spacing (ft)	Pile Shear Strength (lbs)
Soldier Pile	🟪	8	160000

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)	Water Surface
Fill/Colluvium	🟨	110	0	30	Water Surface
Weathered Pre-Olympia Nonglacial	🟩	125	0	35	Water Surface
Coarse-Grained Pre-Olympia Nonglacial	🟧	135	0	40	Water Surface
Fine-Grained Pre-Olympia Nonglacial	🟪	130	500	30	None

Legend

- ⬚ Search Grid
- ⧓ Search Limits
- ▼ Modeled Groundwater Level
- B-2 Boring Location and Depth

Section E-E'
Seismic

Global Stability Analysis

Geotechnical Engineering Report
Buttenwieser/Wiley Residence
Mercer Island, WA

SLIDEINTERPRET 8.032

SCALE: 1:250

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PROJECT NO.
200631

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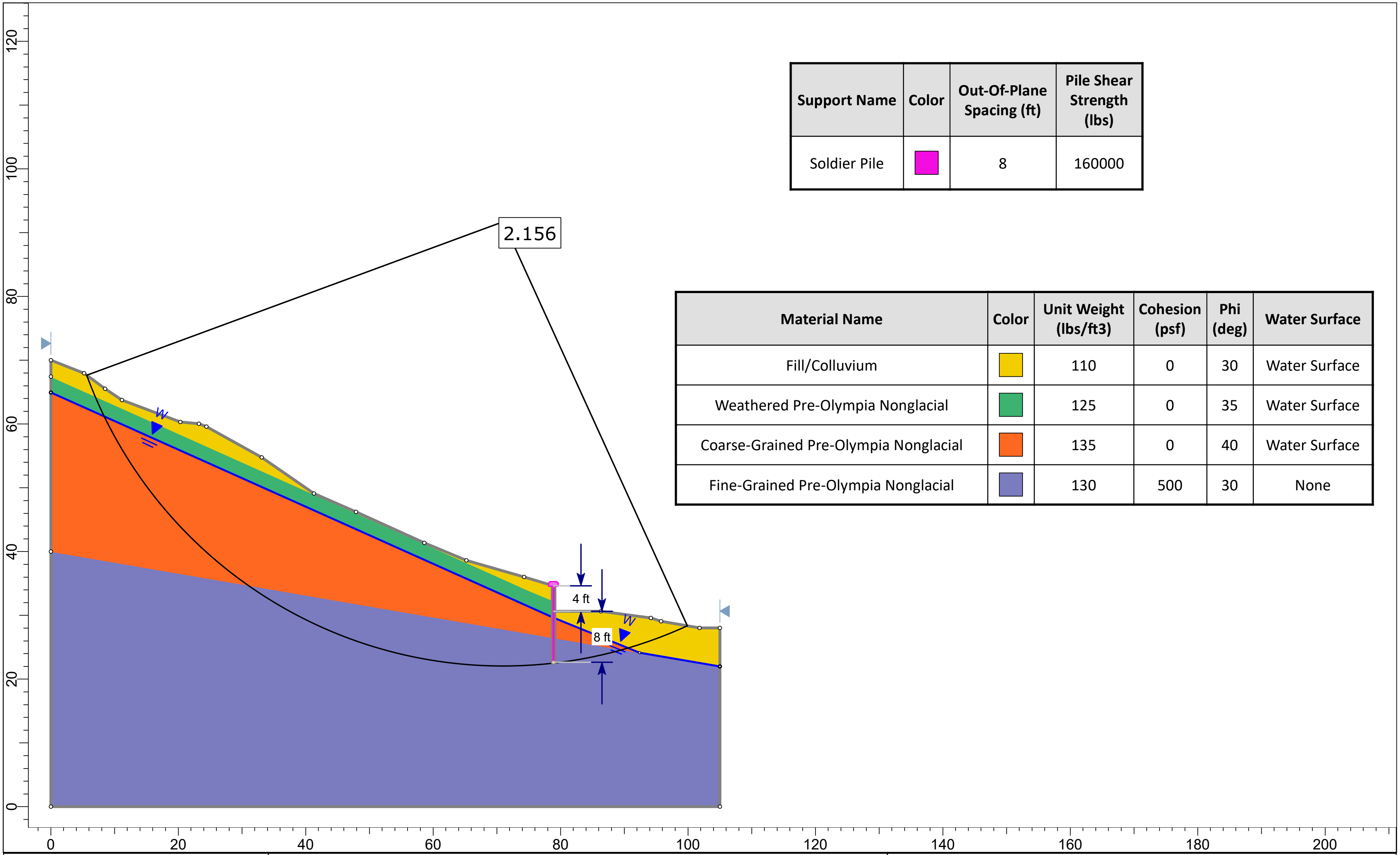
APPENDIX:
C-10



Support Name	Color	Out-Of-Plane Spacing (ft)	Pile Shear Strength (lbs)
Soldier Pile	🟪	8	160000

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)	Water Surface
Fill/Colluvium	🟨	110	0	30	Water Surface
Weathered Pre-Olympia Nonglacial	🟩	125	0	35	Water Surface
Coarse-Grained Pre-Olympia Nonglacial	🟧	135	0	40	Water Surface
Fine-Grained Pre-Olympia Nonglacial	🟦	130	500	30	None

Legend

- ⬚ Search Grid
- ⧓ Search Limits
- ▼ Modeled Groundwater Level
- B-2 Boring Location and Depth

Section E-E'
Static

Global Stability Analysis

Geotechnical Engineering Report
Buttenwieser/Wiley Residence
Mercer Island, WA

SLIDEINTERPRET 8.032

SCALE: 1:250

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APPENDIX:
C-11

APPENDIX D

Report Limitations and Guidelines for Use

REPORT LIMITATIONS AND GUIDELINES FOR USE

Geoscience is Not Exact

The geoscience practices (geotechnical engineering, geology, and environmental science) are far less exact than other engineering and natural science disciplines. It is important to recognize this limitation in evaluating the content of the report. If you are unclear how these "Report Limitations and Guidelines for Use" apply to your project or property, you should contact Aspect Consulting, LLC (Aspect).

This Report and Project-Specific Factors

Aspect’s services are designed to meet the specific needs of our clients. Aspect has performed the services in general accordance with our agreement (the Agreement) with the Client (defined under the Limitations section of this project’s work product). This report has been prepared for the exclusive use of the Client. This report should not be applied for any purpose or project except the purpose described in the Agreement.

Aspect considered many unique, project-specific factors when establishing the Scope of Work for this project and report. You should not rely on this report if it was:

- Not prepared for you;
- Not prepared for the specific purpose identified in the Agreement;
- Not prepared for the specific subject property assessed; or
- Completed before important changes occurred concerning the subject property, project, or governmental regulatory actions.

If changes are made to the project or subject property after the date of this report, Aspect should be retained to assess the impact of the changes with respect to the conclusions contained in the report.

Reliance Conditions for Third Parties

This report was prepared for the exclusive use of the Client. No other party may rely on the product of our services unless we agree in advance to such reliance in writing. This is to provide our firm with reasonable protection against liability claims by third parties with whom there would otherwise be no contractual limitations. Within the limitations of scope, schedule, and budget, our services have been executed in accordance with our Agreement with the Client and recognized geoscience practices in the same locality and involving similar conditions at the time this report was prepared

Property Conditions Change Over Time

This report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by events such as a change in property use or occupancy, or by natural events, such as floods, earthquakes, slope instability, or groundwater fluctuations. If any of the described events may have occurred following the issuance of the report, you should contact Aspect so that we may evaluate whether changed conditions affect the continued reliability or applicability of our conclusions and recommendations.

Geotechnical, Geologic, and Environmental Reports Are Not Interchangeable

The equipment, techniques, and personnel used to perform a geotechnical or geologic study differ significantly from those used to perform an environmental study and vice versa. For that reason, a geotechnical engineering or geologic report does not usually address any environmental findings, conclusions, or recommendations (e.g., about the likelihood of encountering underground storage tanks or regulated contaminants). Similarly, environmental reports are not used to address geotechnical or geologic concerns regarding the subject property.

We appreciate the opportunity to perform these services. If you have any questions please contact the Aspect Project Manager for this project.